# ENGINEERING NEWS
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## AMERICAN RAILWAY JOURNAL.

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THE HEPBURN NICARAGUA CANAL BILL passed the House of Representatives on Jan. 9 by a vote of 302 to 2. Previous to its final passage a vote was taken on an amendment offered by Mr. Shackleford of Missouri, which authorized the President to make the final choice between the Nicaragua and Panama routes, provided the property and franchises of the Panama company could be secured for $40,000,000. The amendment was defeated by a vote of 162 to 170. The bill as passed authorizes the President to obtain from Nicaragua and Costa Rica control of the necessary territory and to employ such persons in constructing the canal as may seem to him wise.

THE BUSINESS ON THE NEW YORK STATE canals, in 1901, is reported upon by Col. J. N. Partridge, lately Superintendent of Public Works, as follows: The Black River Canal was opened on June 1, and all the other canals on May 1, 1901; they were closed on Nov. 30. The total tonnage moved on all canals amounted to 3,420,613 tons, as compared with 3,345,941 tons in 1900. Of this total 2,276,199 tons went eastward, and 1,144,414 tons westward; there were 1,312,526 tons of through and 2,108,087 tons of way freight. For maintenance and ordinary repairs, for the year ending Sept. 30, 1901, the sum of $972,633 was expended. In all 36 pieces of public work were provided for by special appropriations, and of these 12 were commenced by contract and 12 were being executed by State forces. The contract work amounts to $152,369; and of the State work eight pieces have been completed at a cost of $18,697, with an appropriation of $19,500. Colonel Partridge points out the necessity of continuing to make provision for extraordinary repair and improvement of existing structures. The usual annual appropriation of $350,000 for this purpose was reduced to $325,000 last year; but he only asks for $275,000 for the present year for extraordinary repairs.

A NEW TYPE of light-draft Mississippi steamboat is being built by a company organized for this purpose in New Orleans. These boats are designed to carry from 40 to 60 tons of freight, on a maximum draft of 2½ ft. at a speed of 10 to 15 miles per hour. They can handle cargo over the stern as well as bow, eliminating the necessity for turning at landings. The hull will be built of steel, and each boat will be 125 ft. long, 25 ft. wide, 6 ft. deep. Each boat will have two three-stage compound engines, operating two gun-metal propellers with removable blades, working in tunnels formed in the hull. The claim is made that these boats will only use half the coal of the ordinary type of river steamboat; that much time will be saved in the landings, and that they will profitably navigate streams too shallow for ordinary service. The type is not an untried one, as it is similar to the boats used by the Egyptian and Argentine governments; and the United States is building one for use on the Great Kanawha River improvement work.

THE DRAINAGE OF THE EVERGLADES, FLA., is contemplated in a land-deal now in progress involving $5,000,000. The Florida East Coast Drainage & Sugar Co. controls 1,000,000 acres in the Everglades region of Florida, parts of which have been drained, and the land found excellent for growing sugar cane. This company now proposes to drain more of this land and ultimately to produce 2,000,000 tons of sugar per year. Moritz Fitchenberg and Henry Benedict, of New York, are said to be financing this scheme.

LEAGUE ISLAND PARK, Philadelphia, is a large area of swamp land, lying below high-tide mark, at the south end of "The Neck," in that city and near the League Island Navy Yard. Under a previous contract a lake of 12 acres in area was dug in this territory west of Broad St., and 25 acres of dry land were secured with this excavated material, at a cost of $125,000. Under a recent contract made with the city of Philadelphia, the American Dredging Co. has obligated itself to give for $250,000, one mile of 26-ft. channel in the Delaware River, 1½ miles of 26-ft. channel in the Schuylkill River, and 100 acres of dry land in the Park. The mud and gravel from the two river channels are to be deposited in a dumping basin, 600 × 200 ft. and 18 ft. deep, formed in the Back Channel, in the rear of the Navy Yard. From this basin a large hydraulic pump will lift the diluted mud and pour it over the lowland east of Broad St.; the limits of the fill being fixed by 8 ft. embankments, 12,600 ft. in length. The fill is to be 8 ft. above the present land level.

THE MANILA HARBOR IMPROVEMENTS contemplates a harbor with 30 ft. depth of water; the deepening of the channel of the Pasig River to 16 ft., and the incidental reclamation of 150 acres of land in the city, to be sold by the government. The report of the Philippine Commission estimates two years' time and $2,150,000 as required for completing this work. The Puget Sound Bridge & Dredging Co. has already secured the contract for the bulk of this work, and is now on the ground.

THE NILE IS CONTROLLED BY ABYSSINIA, says Mr. Cope Whitehouse in the London "Times." He explains that $^{15}/_{17}$ths of the entire summer supply of the Nile—between March and July—comes from Lake Tsana, or Dembea, in the interior of Abyssinia. This lake is little known, but is described by a French explorer as covering 2,400 sq. miles, with only one outlet into the Blue Nile. Mr. Whitehouse goes on to say that a temporary barrier, 2 or 3 ft. high at this outlet, would hold back nearly ten times the amount of water now reaching Egypt during the dry season; and if this barrier existed for even two months the Nile would be so reduced in volume that no water would reach Lower Egypt. All of which is in favor of the construction of the Wadi Raiyan storage reservoir, proposed by Mr. Whitehouse.

THE MOST SERIOUS RAILWAY ACCIDENT of the week was a butting collision between an eastbound express passenger train and a westbound freight on the Norfolk & Western Ry., near Gray, W. Va., on Jan. 7, 1902. Three men were killed in the wreck and a number injured. The freight train had a flagman out to stop the approaching passenger train, but his signals were not noticed and he himself was run over and killed, the collision following a moment later.

THE COLLISION IN THE NEW YORK CENTRAL tunnel, near 56th St., New York city, reported in our last issue, resulted in the death of 16 persons, and the injury of about 40 others. Of the killed, 15 either were killed outright or died within a few minutes; one of those injured and taken to the hospital died after a few days. Several of the remaining injured are still in a precarious condition. Inquiries into the cause of the wreck are being conducted by several parties; a coroner's inquest is being held, the state railroad commissioners are investigating the wreck on their part, and the district attorney of New York has also taken up the matter. These different inquiries are expected to thoroughly fix the responsibility for the accident. As far as is known up to the present, the engineer of the White Plains train, which ran into the rear of the Danbury train, failed to see either distant or home signal set against him; his failure to do so seems to be largely due to the condition of the tunnel, which was filled with smoke and steam.

THE 1901 FIRE LOSS IN THE UNITED STATES and Canada was $164,347,450, or, about one million more than in 1900, and about 27 millions more than in 1899. During 1901 there were 2,522 fires destroying more than $10,000 in value; the Jacksonville, Fla., conflagration destroyed property worth $10,565,000; and the burning of the Montreal Board of Trade building involved a loss of $2,750,000. loss in a tabular form, classified by months, and by values of property destroyed.

THE LOWER TALLASSEE DAM, some 30 miles from Montgomery, Ala., which was damaged by flood on Dec. 29, 1901, was in an unfinished condition and still in the hands of the contractor. The same was also true of the upper Tallassee Dam, as is stated in a letter by the International Hydraulic Co., engineers for the dam, published elsewhere in this issue. Both dams were being built across the Tallapoosa River, the upper one, as stated in the lengthy description of it published in our issue of Dec. 5, 1901, for the Montgomery Water Power Co., and the lower one for the Tallassee Falls Manufacturing Co. The engineers for the lower dam were Alber & Byrne, of Birmingham, Ala., and Mr. B. O. Watkins was the contractor.

BULKHEAD DOORS CLOSED FROM THE BRIDGE have been installed on the North German-Lloyd steamer "Kronprinz Wilhelm," and, it is stated, will be applied as rapidly as practicable to all the vessels of the company. The system employed is known as the Dörr System in which the doors are operated by hydraulic power.

OCEAN PASSENGER TRAFFIC, in 1901, is reported upon by Mr. Wm. C. Moore, landing agent at Ellis Island, N.Y. harbor. The steamers of all lines made 887 trips, and they carried 128,143 cabin passengers and 438,868 steerage passengers. These figures in 1900 were 137,852 and 403,491, respectively.

OCEAN COAL TRANSPORT is reported upon by U. S. Consul General R. P. Skinner, of Marseilles, France. The business is now being done at unprofitable rates; the figures being $1.20 to $1.35 per ton from Cardiff to Marseilles; $2.25, from Baltimore or Newport News to Marseilles. The comparative cost of delivering 3,000 tons of coal at Marseilles from Cardiff or Baltimore is set down as follows, making due allowance for the increased length of voyage: From Cardiff, $1.02; from Baltimore, $1.22. But more time is required for the shipment from Baltimore, and a return cargo is uncertain. Standard navigation coal from Cardiff is now selling in Marseilles at $5.34 per ton; it cost $3.16 per ton at Cardiff, with the 24 cts. export tax added. The exact selling price of American coal at this port is not given, but Mr. Skinner says that in 11 months of 1901, 184,797 tons of American coal was sold in Marseilles against 941,171 tons of British coal.

BIDS FOR WATER PURIFICATION WORKS at Pittsburg, Pa., will be received on Feb. 10. The contract will include a receiving and two settling basins, 25 open slow sand filters and 8,000 ft. of 36 to 72-in. steel pipe. Further particulars are given in our advertising columns. Mr. J. Guy McCandless is Director of Public Works; Mr. Morris Knowles, Assoc. M. Am. Soc. C. E., is Resident Engineer of the Bureau of Filtration, and Mr. Allen Hazen, M. Am. Soc. C. E., 220 Broadway, New York city, is Consulting Engineer.

A BANQUET GIVEN IN HONOR OF G. MARCONI for his recent achievement in communicating across the Atlantic by wireless telegraphy was held at the Waldorf-Astoria Hotel in New York on the evening of Jan. 13. The banquet was given by the American Institute of Electrical Engineers. Sig. Marconi spoke briefly in recognition of the honor done him, and expressed himself as confident of soon being able to transmit messages by his system with absolute secrecy. Addresses were also made by President Steinmetz of the Institute, Prof. Elihu Thomson and Dr. M. I. Pupin. Letters of regret were read that had been received from Nicola Tesla, Thomas A. Edison, Mayor Low of New York, and others who had been invited to attend the banquet.

WIRELESS TELEPHONY is said to have been accomplished with an apparatus devised by Mr. Nathan Stubblefield, of Murray, Ky. On Jan. 1 of this year Mr. Stubblefield, according to press dispatches, gave a demonstration of the working of his apparatus before a large number of witnesses. The system was successful up to 500 yds. The precise nature of the transmitting and receiving elements has not been made known; it appears that a special form of battery invented by Mr. Stubblefield played a large part in the wireless transmission of the sound. The inventor is carrying his experiments farther.

THE FIRST TRAIN OPERATED BY ELECTRICITY from the new power house of the Manhattan Railway Co. in New York city was run over the Second Ave. line of that company on the afternoon of Jan. 9. The train was made up of six cars, and was run from the station at South Ferry to the 129th St. station. Over 200 guests had been invited to make the trip by the Manhattan Railway Co. The test run was satisfactory in every respect; no trouble was experienced at any point, and the full speed of 15 miles per hour was reached in half the time required by the present steam-locomotive trains of five cars. Part of the trains on the Second Ave. line are now regularly being run by electricity.

SUPPORTING A TELEPHONE EXCHANGE in mid air while erecting a new building beneath and then around it is reported from Youngstown, O. The telephone company refused to cancel its lease unless paid a large sum for doing so. The owners thereupon shored up the top story of their building, exchange and all, and began to construct the new building. When the new building is up to the second story it is proposed to lower the part of the old building containing the exchange and incorporate it with the new structure. We are indebted to Mr. Benj. H. Flynn, Engineer of the Ohio State Board of Health, Columbus, O., for this information.

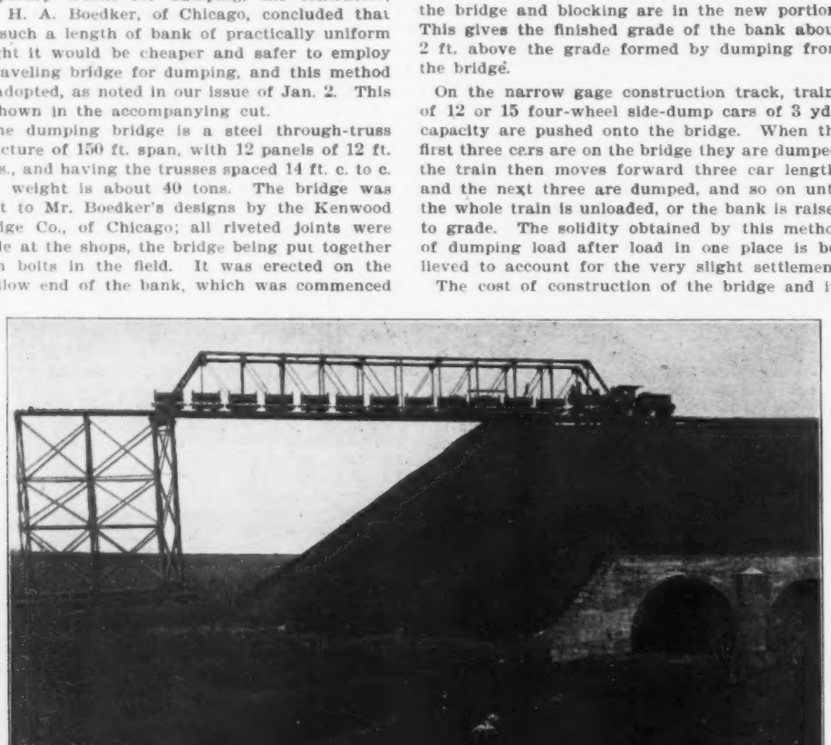
## A NEW METHOD OF BUILDING EMBANKMENTS.

In the improvement of the Iowa division of the main line of the Chicago, Burlington & Quincy R. R., from Chicago to Omaha, there is some heavy embankment work, and one of these banks is being built in a very novel manner. The work in question is on a new piece of line nine miles long, built to avoid a long hill on which pusher engines are now employed. The new line will have a grade of 0.6%. On this line, about two miles east of Ottumwa, Ia., will be a bank about 1,200 ft. long, with a height of 60 to 65 ft. for the greater part of the distance, and crossing the valley of Sugar Creek. Instead of building a temporary trestle for dumping, the contractor, Mr. H. A. Boedker, of Chicago, concluded that for such a length of bank of practically uniform height it would be cheaper and safer to employ a traveling bridge for dumping, and this method he adopted, as noted in our issue of Jan. 2. This is shown in the accompanying cut.

The dumping bridge is a steel through-truss structure of 150 ft. span, with 12 panels of 12 ft. 6 ins., and having the trusses spaced 14 ft. c. to c. The weight is about 40 tons. The bridge was built to Mr. Boedker's designs by the Kenwood Bridge Co., of Chicago; all riveted joints were made at the shops, the bridge being put together with bolts in the field. It was erected on the shallow end of the bank, which was commenced on the tower. This arrangement was adopted for the reason that it was expected the bank would settle, so that the rear end of the bridge would have to be frequently raised or jacked up. In practice, however, there is found to be practically no settlement, owing to the method of dumping, but the pin bearing was convenient in allowing for the inclined grade of the first portion of the travel of the tower. The whole structure is moved forward by a team hauling on a set of blocks and tackle attached to the tower. It is moved 30 ft. at a time, or exactly one rail length. Behind the bridge is a length of timber blocking. When the bridge is ready to be moved, earth is dumped on either side of this, and is filled in when the bridge and blocking are in the new portion. This gives the finished grade of the bank about 2 ft. above the grade formed by dumping from the bridge.

On the narrow gage construction track, trains of 12 or 15 four-wheel side-dump cars of 3 yds. capacity are pushed onto the bridge. When the first three cars are on the bridge they are dumped, the train then moves forward three car lengths and the next three are dumped, and so on until the whole train is unloaded, or the bank is raised to grade. The solidity obtained by this method of dumping load after load in one place is believed to account for the very slight settlement.

The cost of construction of the bridge and its



BUILDING EMBANKMENT BY DUMPING FROM A TRAVELING BRIDGE; CHICAGO, BURLINGTON & QUINCY R. R.

W. L. Breckenridge, Chief Engineer, Chicago, Ill. H. A. Boedker & Co., Contractors, Chicago, Ill.

with teams, and was then run out to the deeper portion on which it was to work. The bridge carries a single track of 3 ft. gage. The floor beams have the stringers riveted between them, and at the rear end of the bridge the bottom lateral bracing is only between the stringers, as the dumping is done at this portion. At the front end of the bridge, where no dumping is done, there is the usual bottom lateral bracing between panel points. The top lateral bracing is continuous.

The front end of the bridge is supported by a timber trestle about 60 ft. high, 25 ft. wide at the base and 16 ft. on top. It has a pier bent of 7 ft. span and two bents of 20 ft. span. The top or deck has a rise of 1 ft. from the bridge, so that when working on the level the dump cars will not have a tendency to run away. This tower is carried by wheels running on two rails 25 ft. apart, laid on timber blocking on a shallow bank built to give the required grade. Work was commenced first at the left of the culvert, from which the tower was carried on a down grade until it reached the level, at which it is to run across the valley. In the illustration, the tower is inclined on account of this grade, but beyond this point will continue its work on the level. To provide against the possibility of severe wind storms, each side of the tower has three cable guys, attached to deadmen.

The rear end of the bridge is supported by timber blocking on top of the bank, with four rollers under each end post to allow for the travel of the structure. The front end rests on pin bearings supports is estimated to have been less than half that of a temporary trestle over the whole length of the bank, and of such a height, while the method of operation is much safer. Besides the liability of accidents on temporary trestles, there is often difficulty in getting men to work upon them. To prevent collapse the filling must be done for a whole train length, and not from one end, and the work of dumping cars from the insecure footing of a trestle deck 60 ft. above the ground is neither easy nor safe. The bridge has one side of the floor open for dumping, while on the other side is a plank platform level with the rails, so that the cars can be dumped easily and rapidly and with entire safety. Even if a car should be dumped with doors closed, so that the load would overbalance it and tip it sideways off the track, it would be caught by the floor beams, while such an occurrence on a trestle might have very serious results.

The material is clay, excavated by a steam shovel, and the haul is now about ¾-mile, but will eventually be about 2 miles. The bank will contain about 300,000 cu. yds., and at present about 300 to 500 cars are dumped, or about 1,600 cu. yds. deposited, per day of 24 hours. The work is delayed, however, by the severe weather, the clay being frozen so that it has to be blasted, and the large lumps handled with chains. With favorable conditions of warm weather, it is believed that from 800 to 1,000 cars can be dumped daily. The moving of the structure, 30 ft. at a time, takes about 3 hours. Work is carried on night and day. From the top lateral bracing of the bridge are suspended two arc lamps, while work at the steam shovel is lighted by clusters of incandescent lamps, the vibration being too great for the satisfactory use of arc lamps.

The bank is finished to a width of 16 ft. at subgrade, but will be widened by side dumping to a width of 40 ft., this being considered necessary to provide for the washing of the material by rains before the bank is consolidated and comes to its final shape. It will carry two tracks, 13 ft. c. to c. Behind the trestle tower, in the accompanying view, is seen the bank of the original line, and this will be crossed by the new bank at an elevation of about 35 ft.

The culvert shown in the view is of monolithic concrete construction, and has two 30-ft. arches, about 220 ft. long. This is to carry the water of Sugar Creek, which stream is dry for the greater part of the year, but has high freshets in the summer. This structure was built by the railway company, which is using concrete very extensively for structures of this class, both large and small.

The contractors for the two-mile section which includes the Sugar Creek embankment, are H. A. Boedker & Co., 1208 Ellsworth Building, Chicago, Ill. This contract covers about 500,000 cu. yds. of earth work, of which about 300,000 yds. will be in this one bank, and the method of building the bank (as above described) was conceived and carried out by Mr. Boedker, the President of the company, to whom we are indebted for information. The work is under the general direction of Mr. W. L. Breckenridge, Chief Engineer; and Mr. Frank Beckwith, of Burlington, Ia., is Division Engineer.

## THE PRESIDENTS OF FOUR AMERICAN ENGINEERING SOCIETIES.

(With Inset Sheet of Portraits.)

In accordance with our annual custom we present in this issue portraits of the Presidents of the four leading national engineering societies of America: the Civil, Mechanical, Mining and Electrical engineers. Brief biographical notes of each of these gentlemen are also presented as follows:

### ROBERT MOORE.

Mr. Robert Moore, who has just been elected President of the American Society of Civil Engineers, was born at New Castle, Pa., June 19, 1838. His father, Mr. Henry C. Moore, was a civil engineer, and his maternal grandfather, Mr. Charles T. Whippo, was for some time Chief Engineer of the Pennsylvania State Canal, between Beaver and Erie, in that State.

When Robert Moore was two years old his father removed to Indiana to accept the position of Chief Engineer of the Whitewater Canal, then under construction; and on the completion of this work the father was employed as an engineer on railway construction in Indiana and Ohio. During his school vacations the son assisted the father on a number of these surveys, as flagman and rodman.

In 1858, Robert Moore was graduated from Miami University, at Oxford, O., and after several years he began the work of a civil engineer; one of his first professional engagements being that of Assistant U. S. Engineer at Camp Nelson, Ky. From that time continuously Mr. Moore has been mainly engaged in the location and construction of railways in the States of Illinois, Indiana, Ohio and Missouri, with his home in St. Louis, Mo.

Mr. Moore was chief engineer, for two years, of a road which is now part of the Baltimore & Ohio Southwestern R. R. In 1868-9, he was Chief Engineer of the Terre Haute and Indianapolis R. R., and in 1869-70, as chief engineer, he built a railway from Belleville to Duquoin, Ill., which now forms a part of the Illinois Central system. He then assisted his father in completing a railway from Pleasant Hill, Mo., to Lawrence, Kansas. This road has since become part of the main line of the Atchison, Topeka & Santa Fe Ry. In 1872-3, Mr. Moore completed a railway from Lorain to Urichsville, O., which is now included in the Cleveland & Wheeling Ry.; and after this he spent some time in miscellaneous work, including the location of the eastern half of the Indiana, Decatur & Western Ry.

In 1877, Mr. Moore was appointed Sewer Commissioner and a member of the Board of Public Improvements of the City of St. Louis, Mo., and over this board Mr. Henry Flad, Past-President of the American Society of Civil Engineers, presided. Mr. Moore resigned from this position in 1881 to resume railway work, and he has since then conducted general practice as a constructing and consulting engineer, with office in St. Louis. During this time he has built several short lines of railway, including the St. Louis, Peoria & Northern R. R. from Peoria, Ill., to St. Louis, Mo. This road now belongs in part to the Illinois Central system, and in part to the Chicago & Alton. He served as Chief Engineer of the St. Louis Merchants' Bridge Terminal Ry. Co., and the elevated viaduct of this company was built under his direction.

He has also acted as Consulting Engineer for the St. Louis & Illinois Bridge Co., owning the Eads bridge; the St. Paul & Duluth Ry. Co.; the Toledo, Peoria & Western Ry. Co.; the Reorganization Committee of the St. Louis & Southwestern Ry., and the Reorganization Committee of the Atchison, Topeka & Santa Fe Ry.

In 1896, Mr. Moore represented the Los Angeles Terminal Ry. in the harbor controversy that resulted in the selection of San Pedro as the deep water harbor on the Southern California coast. In 1897 he was a member of the Brazos River Board, which reported to Congress upon the works at the mouth of the Brazos River and their value to the government. In 1899-1900 he was a member of the Southwest Pass Board, which reported to Congress a plan and estimates for deepening to 35 ft. the Southwest Pass at the delta of the Mississippi River.

At the present time Mr. Moore is Chief Engineer on the construction of the Southern Missouri Ry., and is also Consulting Engineer for the Terminal Railway Association of St. Louis, operating the two bridges over the Mississippi River, at that place, and for several other railway companies.

Mr. Moore was elected a Member of the American Society of Civil Engineers on April 5, 1876, and a Member of the Institution of Civil Engineers on Dec. 6, 1887. He has twice been President of the Engineers' Club of St. Louis, and he is a member of the Board of Education of St. Louis, and a Past-President of the Academy of Science of that city.

## EDWIN REYNOLDS.

The new President of the American Society of Mechanical Engineers, whose election was announced at the recent annual meeting of the Society in New York city, is Mr. Edwin Reynolds, of Milwaukee, Wis. Few men, who have attained such eminence in the enginering profession, have risen from humbler beginnings, or have had less to aid their progress, aside from their own energy and ability, than Mr. Reynolds.

He was born nearly 71 years ago, in Mansfield, Conn., a small farming community in the northeastern part of the State. He was next to the youngest in a family of 12 children, and his early life was like that of most New England farmer's boys a half century ago. In summer he worked on the farm, and in the winter attended the district school. In his sixteenth summer he was persuaded to leave the farm and learn the machinists' trade, and was apprenticed for three years to Anson P. Kinney, the owner of a small general machinery and repair shop, which made something of a specialty of textile machinery. The young apprentice lived in his employer's family, and his wages were for the first year "$30 and found;" for the second year, $45, and for the third $60.

In 1850, at the age of 19, Mr. Reynolds started out as a journeyman machinist and spent a year in the employ of Smith, Winchester & Co., of South Windham, Conn., builders of paper machinery. The next year he went to the Woodruff & Beach Iron Works, at Hartford, one of the principal builders of steam engines of that day. This appears to be the first time when Mr. Reynolds came in touch with the field of steam engineering, in which he was in after years to do such important work. His work while in this firm's employ, however, was generally directed in another line. Woodruff & Beach were also builders of stone-dressing machinery, and Mr. Reynolds was soon put at the head of a gang on this work, and later was put in charge of all work of this class, superintending both construction and erection.

In 1858 an opportunity came for Mr. Reynolds to go west. A Mr. Stedman, of Aurora, Ind., came to Hartford looking for a superintendent for his machine shop at Aurora, and Mr. Reynolds was recommended for the position by his employer. The shop at Aurora carried on a general line of business, but its work was mostly on farm and plantation machinery, pumps, etc. Mr. Reynolds remained at Aurora two years, when the breaking out of the Civil War had so disastrous an effect on the firm's business that Mr. Reynolds gave up his position and returned to the East, and for seven years made his headquarters in Boston. During this time he was engaged in the development of various lines of special machinery, and also had charge of a shop owned by Geo. T. McLaughlin.

It was in 1867, at the age of 36, that Mr. Reynolds began work in the field of engineering, in which he was to become most prominent, the construction of steam engines. In that year he became a salesman and general engineer for the Corliss Steam Engine Co., of Providence, R. I., at that time the most prominent builders of steam engines in the United States. After four and a half years Mr. Reynolds was made General Superintendent of the Corliss works, and remained there until 1877.

In that year, at the age of 46, he resigned his position with the Corliss Co. and went to Milwaukee to take charge, as engineer, of the E. P. Allis works in that city. To any one taking a superficial view at that day, Mr. Reynolds' change of position must have seemed a piece of folly. The Corliss works was a world-famous and prosperous institution, which had made its proprietor wealthy. The Allis concern was practically unknown; it was located in the far West, where, at that time very little manufacturing industry existed, and that for the most part of a crude order. Its plant consisted of a foundry built to make cast-iron pipe, and a small and poorly-equipped machine shop.

And yet notwithstanding this contrast, it is not unlikely that Mr. Reynolds had the keen foresight to see on the one hand that the day of the Corliss works was passed. That its geographical location and the dictatorial methods of its founder were certain, eventually, to work its ruin. On the other hand, he perceived too, we may presume, that the plant in Milwaukee was capable, under proper management, of becoming an important and prosperous concern, although it is doubtful if even Mr. Reynolds foresaw the extent of the changes which a brief quarter century could work in the business.

Immediately upon going to Milwaukee Mr. Reynolds started the Allis works into the field of steam engine manufacture. He designed the Reynolds-Corliss engine, and it attained almost at once a wide popularity, and put the works on a profit-making basis. Nowhere is Mr. Reynolds' genius as an engineer and business director better illustrated than in the designing of the first girder-frame engine, as it was called. The frame was cast in two parts, since the foundry equipment could not handle a single casting of the necessary size. It was made of a very simple form, with only a single core, because the molders available in the foundry were not skilful enough to tackle a difficult casting. In short, Mr. Reynolds designed not the engine he wanted to build, but the best engine which he could build with the existing tools in the shop. Later, it became necessary to build an engine with frame larger than the foundry could handle at all, and to meet this Mr. Reynolds designed an engine with a wrought-iron frame. With these two designs of engine Mr. Reynolds tided his concern over the period of inadequate shop facilities until he brought out his "1890" horizontal engine, with a design of frame which has been extensively copied by engine builders all over the world.

To enter fully into the enginering achievements of the Allis works under Mr. Reynolds' direction would be, of course, quite beyond the limits of our space, and we can only select two or three notable examples as illustration of Mr. Reynolds' engineering successes.

The Tamarack copper mine in Northern Michigan desired some new stamps, and as Mr. Reynolds had done some work previously for Capt. John Daniells, the manager of the mine, the Allis Co. was asked to bid on the stamps. Mr. Reynolds had never seen a stamp, so he concluded before taking hold of the work, to go up to the mine and see the stamps in operation, and prepare himself for work of this character. He arrived on the ground in company with Mr. Emerson, Superintendent of the stamp mill, and they went out to take a look at the mill. The noise, of course, was something fearful, and after some ten or fifteen minutes had passed in looking at the machinery, they came out, and Mr. Emerson asked him what he thought of their stamp mill. Mr. Reynolds replied: "I would never put in a stamp like that," to which Mr. Emerson replied: "Why not? What is the matter with that?" Mr. Reynolds' answer was: "Well, if I was trying to crush rock I would not start in to do it on a feather bed." This of course referred to the huge substructure of spring timbers and rubber sheeting placed below the anvil of the stamp in order to produce a certain tremor and vibratory action, which was supposed to add to the efficiency of the stamps. What Mr. Reynolds wished to do was to replace all of these spring timbers with a solid base of iron, weighing at least 50 to 75 tons, so that the force of the blow of the stamp would be entirely utilized in crushing the rock.

So convinced was Mr. Reynolds of the correctness of his position, that he went on to Boston to have a conference with the President of the Tamarack Co., with whom he was acquainted. But all his efforts to induce them to allow him to make a change in the design of the stamp were fruitless. They urged that the stamp as built represented standard practice extending back for forty years; that the spring timbers aided materially in pulverizing the rock, and they would not listen to any change. Mr. Reynolds, however, was persistent, and finally secured a promise that when they wanted stamps in the future, they would let him know sufficiently in advance so that he could prepare a design in accordance with his ideas. The first stamps, therefore, built by Mr. Reynolds, were in exact accordance with the previous practice of the Tamarack Co.

In a couple of years came a proposition to bid on additional stamps. Mr. Reynolds made no reply to the letter, but packed his grip and answered it in person in Boston, and reminded them of their promise to let him design a stamp as he believed it ought to be designed. After a hard fight he obtained an agreement substantially as follows: The stamp itself was to be in every particular a duplicate of those previously built, including the bolt holes and means of securing it to a spring timber foundation. It was to be placed, however, on three 10-in. iron plates planed and fitted together, the anvil block also being planed and resting directly on the plates. There were to be no rubber or spring timbers in the foundation. If the 40 tons of iron, which was the amount they agreed to let him use as a trial, was retained, the Tamarack Co. was to pay for it at the rate of about 3 cts. per lb.. If the scheme did not prove to be an improvement, this iron was to be removed and Mr. Reynolds and the Tamarack Company were to share the loss equally. So confident were the Tamarack people that removal would be necessary, that they stipulated that the design must be such that the removal of the iron should not cause more than two days delay in the operation of the stamp.

The stamp was finally installed and put in operation, and the next thing that Mr. Reynolds heard from it was a telegram from Mr. Daniells saying that the record of the first week's continuous run was 1,920 tons; 1,200 tons in an equal length of time had been considered a great record for stamps of the old type. The prestige which this victory of Mr. Reynolds gave to the Allis Co. among users of mining machinery can readily be understood.

The production of the Reynolds-Allis blowing engine is another interesting incident. The Allis Co. had done some small work for the Joliet steel

works, and Mr. Reynolds was in Joliet at one time when it was incidentally mentioned that they were preparing to put in a large Bessemer blowing engine. Mr. Reynolds suggested to Mr. H. S. Smith, the general manager, that he would like to have a chance to put in a design and bid. Previous to that time the standard blowing engine had been built with wood-packed piston and hard rubber valves.

When Mr. Reynolds returned with his plans he found himself in competition with three other designs: one prepared by the engineers of the Joliet works; one prepared by Mr. A. L. Holley, and one prepared by a German engineering company. Mr. Reynolds' design was the wrought-iron frame, because at this time he could not build an engine of this size in any other way with the Allis equipment. The air and steam cylinders were tandem, as in his present practice, but the valve gear was in every way a radical departure. Iron packing rings were used, and the valves were of pressed steel and of the cup form now so commonly employed.

There was a battle royal over the plans, and after the matter had been discussed pro and con, Mr. Smith came out with the statement that for his part he considered Mr. Reynolds' design the best of those submitted, and eventually the contract was given to the Allis Company.

The performance of the engine justified all that had been expected of it, and steel men from all over the country came to look at the new blowing engine. From that one engine developed the business in blowing engines which has been one of the important departments of the Allis shops. Mr. Carnegie within a few months after the Joliet engine was in operation, gave an order for several more of similar construction, and this was the first of a series of orders from the Carnegie works aggregating probably five million dollars.

The achievements of the Allis Co. in the pumping engine business are familiar to many readers of Enginering News. Practically the business began with the building of a high-duty engine for the Milwaukee Water Works not many years ago. The high record made by this engine brought the company orders, and the large pumping engine business of the company has been the final outgrowth.

The most recent notable example of Mr. Reynolds' work are the engines for the Manhattan Ry. powerhouse in New York city. The plans of the Manhattan Co. called for 1 HP. per sq. ft. of station area, which seemed impossible unless units of 10,000 to 15,000 HP. were adopted. The electrical requirements were such that with a cross-compound engine of this power the flywheel required would have a weight not far from 700,000 lbs. To carry such a weight on two bearings seemed practically out of the question, while to resort to the marine type of engine with its multiplicity of bearings was, from an engineering standpoint, equally objectionable. Mr. Reynolds met the situation by designing an engine with a short shaft and two bearings, the generator being placed between the bearings. At each end of the shaft there was a disk and a single crank pin. To each crank pin was attached a horizontal high-pressure cylinder, and a vertical low-pressure cylinder, the two crank pins standing at an angle of 135° one with the other. With this exceedingly simple arrangement the necessary weight of flywheel was at once cut in half. This design was accepted and ordered by the Manhattan Co., and before the first engine of this type had been put in operation, orders had been taken for over $2,000,000 worth of engines of this type, all of the largest size.

Mr. Reynolds' time at the present writing is very largely taken up with the building of the new shops at West Allis, the plans of which were published in Engineering News of May 23. The portion now under contract and in process of erection will cover nearly 50 acres of ground, and will cost upwards of $3,000,000. The design of these shops is believed by Mr. Reynolds' friends to be one of the best pieces of work with which his name has ever been connected.

Mr. Reynolds was given the honorary degree of LL.D. by the University of Wisconsin, and his name has been placed upon the frieze of the new engineering building of that institution at Madison.

Mr. Reynolds became a member of the American Society of Mechanical Engineers in 1884, and served as a Vice-President of the Society from 1892 to 1894. He is President of the German-American Bank of Milwaukee.

EBEN E. OLCOTT.

Mr. Eben E. Olcott, who was elected President of the American Institute of Mining Engineers, at the annual meeting in February, 1901, is a typical example of the modern kind of mining engineer, the technical graduate who supplements his college course by many years of varied practice before opening an office as consulting engineer.

He was born in New York city in 1854, and received his education in the New York public schools, and later at the College of the City of New York and the School of Mines of Columbia College, now Columbia University. He graduated from the School of Mines with the degree of Engineer of Mines in 1874. His first position, which he held for a year after his graduation, was that of chemist in charge of the Hunt & Douglas process at a copper works in North Carolina; for another year he was Assistant Superintendent of the Pennsylvania Lead Works, in Pittsburg. From 1876 to 1879 he was superintendent of gold mines in Venezuela. A year later he held a similar position in San Juan county, Colorado, and then spent a year traveling as consulting engineer for New York capitalists, visiting and reporting on mines in Colorado, Utah, Nevada and California.

In 1881 he became manager of the St. Helena gold mines, in Las Delicias, Sonora, Mexico, where he spent several years. In 1885 he opened an office in New York as consulting engineer, subsequently forming the firm of Olcott, Fearn & Peele, now Olcott, Corning & Peele, with offices at No. 36 Wall St. In his work as consulting engineer he has visited and reported on many mines and works in the United States, Mexico and South America, including the gold fields of Dutch Guiana, the Sandia and Carrabaya gold region, Peru, the Huantajaya silver mines, Iquique, Chile. In 1886 he was a member of the Cerro de Pasco Mining Commission, reporting on the famous silver mines of Cerro de Pasco, Peru.

In November, 1901, Mr. Olcott had the rare and pleasant experience of being the official head of the party of 170 excursionists, members of the American Institute of Mining Engineers and their ladies, who made the trip through Mexico, which was recently described in our columns. His familiarity with the Spanish language and his wide acquaintance in Mexico were invaluable to the party.

Mr. Olcott was elected to membership in the American Institute of Mining Engineers, in 1874. He has also been a member of the American Society of Civil Engineers since 1893.

CHARLES PROTEUS STEINMETZ.

Mr. Charles Proteus Steinmetz who, last spring was elected President of the American Institute of Electrical Engineers, has for a number of years been one of the foremost workers in the field of electrical engineering in this country. Active in both direct current and alternating current work, he has achieved the most brilliant results in the latter field: it is on his remarkable contributions to the theory of alternating current phenomena that his fame principally rests.

Charles Proteus Steinmetz was born in Breslau, in the Prussian province of Silesia, in 1865. His early education was obtained in the schools of that city, and he also, after graduation from the "gymnasium," or high school, matriculated at the University of Breslau. Here he made his special study mathematics; under the guidance of Prof. Schroeter, a world's authority on synthetic geometry, he became familiar with the most advanced phases of modern mathematical study.

His interest in his chosen study continued after he left the university; he became instructor in mathematics, first at Breslau, and later at Berlin, and devoted himself outside of his mathematical work to physics, chemistry, and allied natural sciences.

A little later the unsettled political conditions of his native country caused Mr. Steinmetz to leave Germany for Austria, and shortly afterward he went to Switzerland. In that country he again entered student life, taking up the study of mechanical engineering at the famous Polytechnic School of Zürich. Later still he removed to France, and from there came to America in 1889.

Mr. Steinmetz' first connection in this country was with the Eickemeyer Co., of Yonkers, N. Y., almost immediately he threw himself into the investigation of electrical phenomena, and with his thorough and many-sided training he rapidly forged ahead in this field. In 1893 he left the Eickemeyer firm for the General Electric Co., with whom he has been continuously since that time, first at Lynn, Mass., and later at Schenectady, N. Y.

The writings of Mr. Steinmetz have been numerous. Among his earlier works are a popular handbook of astronomy, published in Germany, and various articles contributed to the "Zeitschrift für Mathematik und Physik," and the "American Journal of Mathematics" on subjects such as "Theory of Synthetical Geometry," "On Twisted Curves," etc. His work in electrical engineering was almost all done in this country, since entering practical work in that field. His published papers have covered a large range of subjects, most of them in connection with alternating current work. A great part were presented before the American Institute of Electrical Engineers, and form an exceedingly valuable portion of the transactions of that society. Among these contributions are a series of papers "On the Law of Hysteresis," embodying the results of a very extended investigation of the subject; a number of papers on alternating current motors, a piece of mechanism with whose development the name of Mr. Steinmetz will always be coupled; several papers on dielectrics, and probably most important of all, several papers on the application of vector analysis to alternating current problems, the first of which was read before the International Electrical Congress at Chicago, in 1893, and aroused great interest among the scientists and engineers assembled there. Mr. Steinmetz has also written several books on electrical subjects. His "Theory and Calculation of Alternating Current Phenomena," 1897, second edition, 1900, a model of systematic and logical development, and his "Theoretical Elements of Electrical Engineering," which covers the whole field of the theory of electrical phenomena in the same thorough and scientifically clear manner, Mr. Steinmetz himself considers among his best works.

## CHIEF ENGINEER HILL'S REPORT ON PROPOSED CHANGES IN THE NEW CROTON DAM AND JEROME PARK RESERVOIR.

Relating to the report (see Engineering News, Nov. 28, 1901) of the Board of Engineers appointed June 21, 1901, to consider the plans of the earth embankments at the New Croton Dam and Jerome Park Reservoir, which was referred to me by your honorable board by resolution of Nov. 22, 1901, I would respectfully report thereon as follows:

THE NEW CROTON DAM.

That part of the report relating to the New Croton Dam explains in detail the examinations that were made by and under the directions of the engineers to aid them in forming the conclusions arrived at. It gave the physical analyses of samples of earthen material taken from the refill on both sides of the core wall and from the puddled embankment, also of the material called hardpan in its natural state before its use in the banks.

Before the analyses were made the materials had been dried several hours at a temperature of 250° F. Its fineness was obtained by passing it through screens of from 4 to 40,000 meshes per sq. in. and the parts per 10,000 by weight. An analysis was also made to determine the loss by evaporation and ignition, and the clay matter, quartz and silicates.

Accompanying the report there is a graphic exhibit of the results of several tests of the material used in the construction of the embankment and the refill, showing the rate of percolation of water through the material, also its stability, together with an additional report of percolation experiments of the same material made at Cornell University.

*Official Report of Mr. Wm. R. Hill, M. Am. Soc. C. E., Chief Engineer of the Croton Aqueduct Commission, submitted Dec. 4, 1901, but not made public until Jan. 10, 1902.

Many test tubes were driven into the embankments of the reservoirs in the upper Croton watershed to ascertain the water level and the slope of saturation in the embankments, and very complete and interesting plans showing the result of this examination were furnished.

Accompanying the report there are photographs of different views of the dam, comparative cross-sections of high earthen dams and other plans and charts.

I concur with the Board of Engineers in recommending that the plans of construction of the southerly end of the New Croton Dam from the end of the present masonry dam to the gate house be modified by the substitution of a masonry structure for the earthen embankment with core wall.

THE JEROME PARK RESERVOIR.

The core wall and embankment of Jerome Park Reservoir, upon which the Board of Engineers have reported, is built along the southerly end and easterly side of the reservoir. Its length is 2,850 ft., of which for 1,200 ft. (at three places) it is built upon sand and balance upon rock.

At the southerly end the wall is built upon quicksand across a depression that terminates at the Harlem River. The depression is 270 ft. wide, and at its deepest point there is 30 ft. of quicksand under the wall and above the rock.

Samples of the sand examined by the Board of Engineers were obtained by driving 4½-in. pipes from the surface of the ground down to the rock near the wall and forcing the sand out of the pipes with jets of water. Such samples would not indicate the true character of the material, as the finer particles would be separated and carried away in the water, leaving only the coarser or heavier particles for examination.

The character of the material under these walls is described by the engineer who was in charge of their construction. In the official diary for the year 1897 are described the difficulties encountered in sinking the pits for the walls because of the presence of quicksand. The following abstracts from the diary refer to the walls along the southerly end and easterly side, where they are built upon sand:

May 18, 1897.—At core wall trench, where the excavation is generally in quicksand, the progress is very slow, as great care must be used. The timbers are continually settling, and this necessitates additional bracing.

May 20, 1897.—In core wall trench quicksand still causes much trouble; large stream of water boils up in bottom, and has caused settlement of the timbers. Bottom also squeezes up there, and has to be weighted with planks, etc.

June 3, 1897.—Are excavating in quicksand and laying masonry as fast as bottom is excavated. Much trouble is encountered in excavating, as quicksand runs in from sides almost as fast as it is excavated.

June 11, 1897.—About 20 ft. of wall was started this afternoon; quicksand bad; neat cement had to be used near sheeting on sides to prevent it from washing in while bottom is being prepared.

June 29, 1897.—About 2 cu. yds. of rubble masonry laid yesterday in bottom in Portland cement 3 to 1 had to be removed to-day because stream of water undermined it and caused it to settle slightly. Continue work of extending masonry northward, but work is extremely difficult because quicksand and water run in almost as fast as the excavation is made.

June 30, 1897.—Ordered Portland cement 2 to 1 mortar used for core wall for bottom work where Portland 3 to 1 has hitherto been used; change ordered on account of heavy flow of water there.

July 7, 1897.—At core wall on east side the quicksand has been covered with masonry nearly to point where grade meets rock. Quicksand has been encountered in core wall trench about Station 90, 8 ft. below the surface.

Sept. 25, 1897.—Noticed two hair cracks in core wall between Stations 97 and 101, about at point where wall leaves and enters on rock foundation.

Nov. 12, 1897.—The hair cracks in masonry mentioned Sept. 25 have opened slightly, and are being pointed over with Portland cement.

Nov. 23, 1897.—Ordered core wall trench to be 14 ft. wide, on account of quicksand in trench, as indicated by borings, etc.

Dec. 30, 1897.—Commenced building core wall between Stations 77 and 77 + 40 on earth foundation; a great deal of water there, and three steam pumps are required to keep it out.

In my opinion, the building of an embankment and core wall for a reservoir upon such material and under the conditions as described is a gross violation of the rules of good practice, which prescribes that a core wall should be built either upon rock or upon solid impervious material.

Relating to the depression at the south end of the reservoir (Station 76 + 20, Fig. 1.—Ed.), the report of the Board of Engineers states that:

The borings made indicate that this depression is filled with a fine sand, which is saturated with water, is stable when confined and cannot run unless exposed to flowing water with a free outlet.

Here it is admitted that a very dangerous condition is existing if there is a free outlet for the water, and later on the report indicates the presence of an outlet in the following language:

The borings made in this vicinity indicate that about 100 ft. south of the reservoir this depression is narrow, having a very contracted passage for the water which may tend to flow southward through the sand with which the depression is filled.

It is stated that the water will flow through the sand away from the site of the reservoir, but intimates that the quantity would be limited because of a very contracted passage through a narrow depression, as indicated by borings. It seems to me that a proper investigation has not been made to justify making any statement as to the width of the depression, as only two borings have been made on a line crossing the depression, and they are 115 ft. apart. One boring near the westerly side of the valley reached rock at a depth of 13 ft. below the bottom of the reservoir; the other reached rock at a depth of 38 ft. I cannot conceive how the passage for the flow of water is in any way contracted at this place, as the channel is very deep and we have no knowledge as to its width.

The report further states:

The character of the topography adjoining the reservoir boundary convinces us that no serious leaks need be apprehended under this section of core wall founded on earth, and that none can possibly occur having sufficient motion to threaten the stability of the embankment in which it stands.

From this it might be inferred that the ground was rising back of the wall, or else what bearing would the character of the topography have upon the subject, or how could it convince anyone that no serious leaks need be apprehended? The wall at this place is built across a depression and the surface of the ground falls away from the reservoir in a valley terminating at the Harlem River.

Further it (the report.—Ed.) says: "It is evident that the ground water has not a free outlet to the south or east." But in referring to the narrow depression it states there is a passage for the water which may tend to flow southward through the sand.

In relation to the character of the material under the walls, the report states:

The test pit and borings made near it show that the material in which this section of core wall is founded consists of a stratum of *very fine sand* extending from a plane 6 or 8 ft. below the natural surface of the ground down to the rock, similar to the thin strata of fine sand in some of the borings near the first section tested at the Kings Bridge Road.

It is also noted in the report that the sand "is stable when confined." There is not, however, any recital that the sand is confined. On the contrary, the report clearly points to an opening through the depression heretofore noted, and it again does so in the general description of the site of the reservoir, as follows:

The reservoir is evidently in a basin of the rock surface, surrounded by a ridge of rock in which there are apparently some depressions through which water may drain to the north, the south and the east.

In view of the sand being unconfined, it must be unstable. And, further, relating to the character of the material under the walls, the report states:

The difficulties encountered in founding it (the wall) indicate the presence there of a fine material which will run when exposed to flowing water.

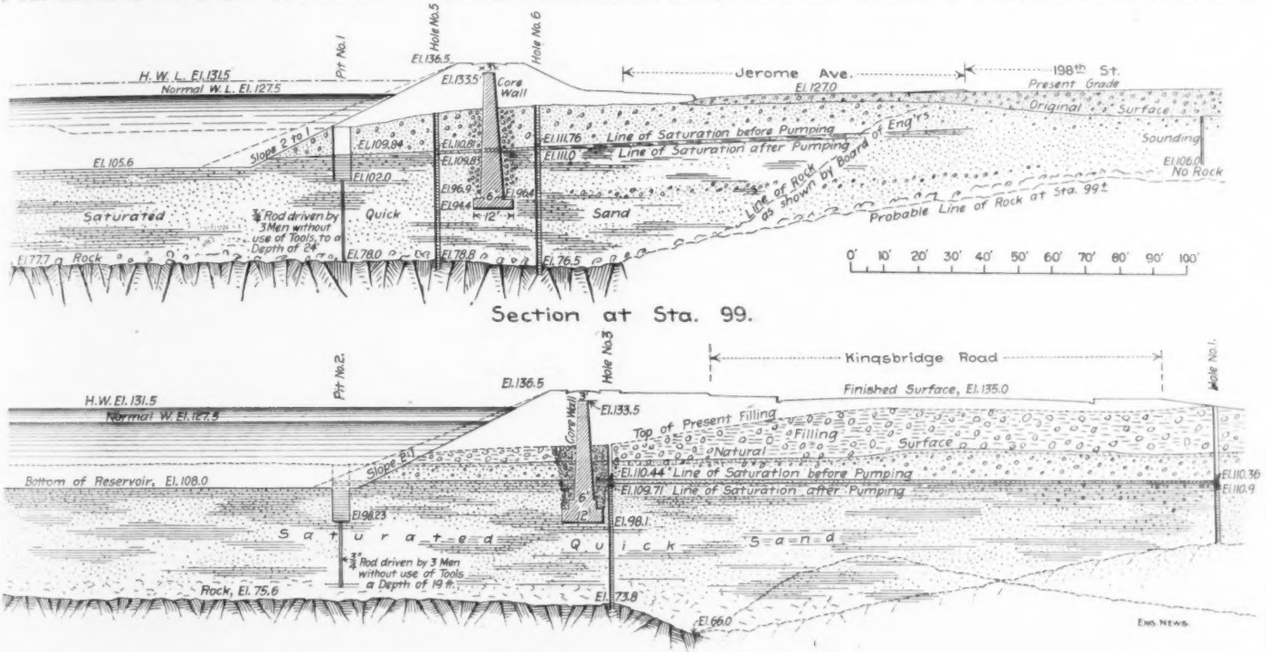
This dangerous condition is still further emphasized by the statement in the report that the water may tend to flow southward through the sand, and further:

We found by numerous tests that all these fine sands are unstable when mechanically agitated in an excess of water, and that they all settle in a firm and compact mass under the water when the agitation ceases. They are quite unlike the true quicksands whose particles are of impalpable fineness and which are quick or unstable under still water.

It is evident to me that in order to determine that there is no quicksand under these walls whose particles are of impalpable fineness that a different method should have been employed in obtaining the samples that were examined other than by forcing the material to the surface by jets of water, thus washing away the finer particles and leaving only the coarser ones for examination.

In excavating pits in the basin of the reservoir near the wall, strata of sand whose particles are of impalpable fineness were found, and samples of such material can be seen in this office, but the danger still exists, whether the sand be coarse or fine.

As to the stability of this material, I would say that three men, without the use of hammers or other tools, drove a ⅞-in. rod through the sand down to the rock, a depth of 19 ft., at the wall at the south end and a depth




Section at Sta. 99.




Section at Sta. 76+20.

FIG. 1. GRAPHIC EXHIBITS OF RESULTS OF TESTS AT JEROME PARK RESERVOIR TO DETERMINE FLOW LINE OF WATER IN THE SAND STRATA UNDERLYING THE EMBANKMENT AND BOTTOM OF THE RESERVOIR.

of 24 ft. at the wall at the easterly side. Hence, I would class this material as being very unstable under any conditions.

I would also call your attention to the abstracts from the diary (as given above.—Ed.) under date of Sept. 25, 1897, noting the appearance of cracks in the wall where it left rock and entered on earth foundation. These cracks were evidently caused by settlement taking place only under that portion of the wall resting on earth. Under date of Nov. 12, 1897, it is stated that these cracks had opened slightly and had been filled with Portland cement mortar. These cracks have opened again, and are now quite pronounced, extending through the wall.

You will notice on the diagram accompanying this report (Fig. 1.—Ed.) that the masonry foundation of the wall is 12 ft. wide, while the base of the wall proper is only 6 ft. wide. This extra width of the foundation was evidently made because of the very loose character of the material upon which the wall is built.

of the wall, thus erroneously showing that the quicksand under and back of the wall is confined in a small pocket by the rock.

The following is, however, noted on the diagram: "Borings show no rock opposite Station 99 + 53 at elevation 106."

This boring was made at a point 210 ft. back of the wall, or 75 ft. beyond where the diagram shows the rock coming to the surface; yet no rock was found by the boring, which was driven to a depth of 1 ft. below the bottom of the reservoir.

Tests were made to determine the extent of saturation and filtration through the sand under this wall in a manner similar to that employed at the south end. The ground was found to be completely saturated with water below a depth of 3 ft. above the finished bottom of the reservoir. The water was pumped from a well inside of the reservoir, and its surface was kept lowered 7 ft. for a period of 25 hours, in which time the water in a

the reservoir bottom the interior lining of concrete should be made double the thickness that is provided for by the present plans.

The above recommendation is made to prevent any sliding or slipping of the inner slope of the embankment in case of the water being rapidly lowered in the reservoir. I cannot anticipate that such trouble would occur unless the bank should become saturated with water by leaking through the concrete. But previously it is stated in the report that the concrete would be almost absolutely water-tight, and yet it seems to me that the recommendation is made because of the fear of leakages through the concrete.

I am of the opinion that there would be a greater liability of the embankment settling than there could be of its sliding, because of the great amount of soft material under its base. The settlement would be most apt to occur when the weight of water would be off the bottom of the reservoir when empty, thus causing an unbalanced

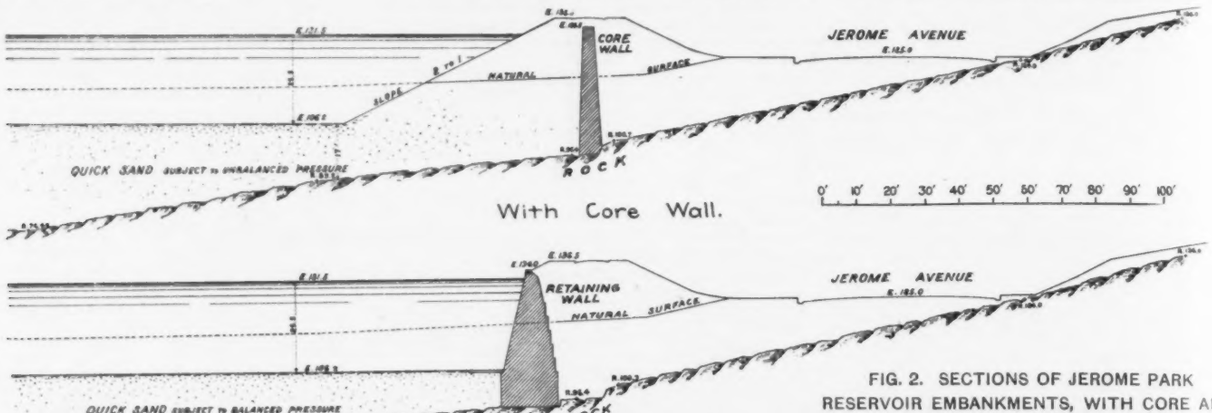



FIG. 2. SECTIONS OF JEROME PARK RESERVOIR EMBANKMENTS, WITH CORE AND RETAINING WALLS, RESPECTIVELY.

(The Lower Section Shows How Unbalanced Pressure Could Have Been Avoided.)

Tests made to determine the leakage through the sand under the wall at the south end were made by sinking a well inside of the basin of the reservoir and by driving several tube wells down to rock outside of the reservoir. While the excavation of the wall inside of the reservoir was in progress the water was pumped from it for a period of 33 hours, during which time the water in the well was gradually lowered 8 ft. as the well was deepened. The effect of this lowered the water 8¾ ins. in a tube well outside of the reservoir near the wall and in two other tube wells located outside of the reservoir, and 265 ft. from the pump the water was lowered 3¼ ins. in each. After pumping was suspended the water rose in all wells. Thus is established the fact that water will percolate and can be drawn a long distance through the sand under and beyond the well, even from a pressure due to only 8 ft., the depth that the water in the well was lowered by pumping. It must be admitted that the rate of percolation would be greatly increased with a reservoir full of water. I would expect serious leaks under this wall as soon as it would be put in use, and that the rate of percolation would increase as time passed. In fact, I would expect that the wall would be ultimately undermined and destroyed.

Relating to the second section of the wall built upon sand, the report describes the adjoining topography as follows:

> Along this section of wall founded on earth the rock outside of the reservoir is evidently, from all the borings which have been made, higher than it is under the embankment, and the natural surface of the ground nowhere within a distance of 500 ft. outside of the core wall is more than 8 ft. below the water surface of the full reservoir.

This is true, but it may be misleading, inasmuch as it might be assumed that the low place, instead of being immediately back of the wall, as is the case, might be 400 to 500 ft. away.

Relating to the third section (Station 99, Fig 1.—Ed.) of wall built upon quicksand, the report states:

> The topographical maps and the records of the borings in this vicinity, furnished us by your engineering department, indicate that the surface of the rock rises rapidly outside this section of the embankment.

The borings furnished by the engineering department do not warrant the statement that the rock rises rapidly. A boring was made at a point 210 ft. back of the wall; it was sunk to the depth of 1 ft. below the bottom of the reservoir and no rock was found. No other borings were made at a greater distance from the wall at this section; hence the depth and slope of the rock are unknown, as far as our records show.

A diagram accompanying the report (not shown by Mr. Hill.—Ed.), showing the results of boring and pumping tests at this point, is also misleading, as it represents the rock as rising to the surface of the ground 135 ft. back

tube well outside of the wall fell 8¾ ins. When pumping was suspended the water rose on both sides of the wall.

It is acknowledged in the report that there will be a small amount of leakage from the reservoir, as is evident from the following abstracts:

> It must be borne in mind that the percolation of water through this bank cannot be rapid enough to induce any movement of the material * * * or that any material amount of water will find its way through or under the walls if they are built in compliance with the specifications. * * * The utmost that can be anticipated is the seepage of a small amount of water through the embankment and the earth, and this would be carried off by the sewers in the adjacent avenues.

I am unable to find in the report any reason that can induce me to agree that the rate of percolation will not be rapid, and that the utmost that can be anticipated is the seepage of a small amount of water. The result of the tests made and the conditions existing strongly confirm the opposite opinion that there would be serious leakages.

Referring to the embankment, the report states:

> That the access of water to the material will be *almost* absolutely cut off by the concrete lining of the banks and bottom of the reservoir.

In my opinion such dependence cannot be placed upon the concrete lining, particularly as there is a bed of quicksand extending over an area of about ten acres under the base of the reservoir. Even assuming that the concrete could be made water-tight, the reservoir might be quickly destroyed by the sinking of a pit for the foundation of a structure outside of the reservoir. The pumping of water from such a pit would draw the water from the saturated material under the concrete lining of the reservoir, thus causing it to settle and crack, and thereby giving free access of water under pressure to the material under the wall through which the water can percolate, as established by the pumping test heretofore mentioned.

I might cite many cases where failures of reservoirs have been attributed to the giving way of the concrete lining. A notable case is the Queens Lane Reservoir, of Philadelphia, which was completed in October, 1894. It was partly filled to a depth of only 10 ft., and then it developed leaks at many points. The rock underlying the site is gneiss and mica; the upper portion is more or less disintegrated; nearly continuous over this is a layer of sandy clay. The bottom and sides were lined with concrete. It cost about $200,000 to repair the defective work.

The only recommendation of the Board of Engineers relating to Jerome Park Reservoir is as follows:

> To preclude all possibility of the sliding or slipping of the inner bank in case of the water being lowered rapidly in the reservoir, we are of the opinion that the inner slope of the reservoir bank should be made 2½ horizontal to 1 vertical, and that at the intersection of this slope with

weight upon the quicksand. The embankment would settle in the soft material, forcing it aside and under the concrete bottom, which in turn would be forced upward and broken, thus affording free access of water to the quicksand under the embankment and walls. I consider that there would also be a liability of the inner slope of the embankment settling, where the core wall is founded on rock. In such a case the difficulty could have been obviated by building a retaining wall with an embankment back of it, thereby dispensing with the inner slope. Diagram No. 3 (see our Fig. 2.—Ed.), illustrating this, accompanies this report.

The flattening of the inner slope of the embankment and an additional thickness of concrete at the toe of the slope, as recommended, would in no way prevent the embankment from settling, or hinder the concrete floor of the reservoir from being forced upward.

In conclusion, I wish to state that in my opinion the material under the embankment and core wall at the southerly end and easterly side of the reservoir at Station 99 is quicksand, having a maximum depth of about 30 ft. under the base of the wall. The sand is unstable and unconfined, and it is completely saturated with water, which is now flowing through it.

In view of the above, I am of the opinion that the embankment and core wall are utterly unfit for the purpose for which they were constructed. I therefore cannot concur in the conclusions and recommendations of the report of the Board of Engineers relating to Jerome Park Reservoir.

## A SPECIAL REPORT ON CHANGES IN THE JEROME PARK RESERVOIR.

In our issue of Nov. 28 we devoted a large amount of space to a report on proposed changes in the new Croton Dam and the Jerome Park Reservoir, now being built for the city of New York under the direction of the Croton Aqueduct Commission, of which Mr. Wm. R. Hill, M. Am. Soc. C. E., is Chief Engineer. The report in question was made by Messrs. J. James R. Croes, Edwin F. Smith and Elnathan Sweet, Mems. Am. Soc. C. E. We have just received from Mr. Hill a report dated Dec. 21, and signed by Mr. Smith, one of the above-named engineers. From this report, and from the report of a meeting of the Aqueduct Commission, held on Dec. 10, 1901, as published in the "City Record" of Jan. 9, 1902, we are enabled to trace the course of events, which has led to the filing of Mr. Smith's report, as follows:

On Nov. 21, or five days after the date of the

Board of Experts' report, Mr. Smith sent a communication to Mr. Hill, which is given below in the opening part of Mr. Smith's report, and which suggested that further examinations be made at the Jerome Park Reservoir, and that modifications of the plans of construction be adopted.

On Dec. 4, Mr. Hill submitted to the Aqueduct Commission the report printed on the preceding pages, 44, 45 and 46, in which he dissented from the conclusions of the experts relating to the proposed changes in the Jerome Park Reservoir. On Dec. 6, the secretary of the Aqueduct Commission sent Mr. Smith a letter, saying that the commissioners agreed with the statement in the last part of his letter of Nov. 27 (see below), and requesting Mr. Smith to

make such careful examination for the purpose of suggesting or recommending such further modifications as may be necessary in the plans as will beyond doubt make the [Jerome Park] reservoir safe, as is stated in your communication above referred to.

It is in response to this letter that Mr. Smith made the report in question, which we give in full below, with the single omission of references to drawings which have not been sent to us:

REPORT BY MR. EDWIN F. SMITH, M. AM. SOC. C. E.*

In compliance with the instructions contained in your communication under date of Dec. 6, I beg to submit the following supplemental report in reference to the Jerome Park Reservoir, now under construction.

The action of the Aqueduct Commissioners on June 21, last, called for the appointment of a Board of Engineers to examine, in addition to the present plans for the construction of the new Croton Dam, "the core-wall and embankments of the Jerome Park Reservoir, of somewhat similar construction, and to report to the Aqueduct Commissioners what changes or modifications, if any, should be made in the plans for the construction of said reservoir."

On Nov. 27, last, in transmitting other papers, I wrote William R. Hill, Chief Engineer Aqueduct Commissioners, as follows:

The report of the Board of Engineers, which is now in your hands, does not deal with any other detail of construction in connection with Jerome Park Reservoir than the stability of the core-wall, and the inner embankment, where they rest upon a foundation of sand, as shown by the exhibits. The only other expression of opinion is contained in a paragraph on page 34 of the report, as follows:

"There is no possible danger of sliding or sloughing of the bank. The utmost that can be anticipated is the seepage of a small amount of water through the embankment and the earth, and this would be carried off by the sewers in the adjacent avenues."

This opinion is based upon the understanding as expressed on page 29 of the report, that the artificial embankment shall be made *"of selected earth,"* and that the bottom of the reservoir shall be covered with concrete *"laid on a surface of good earth well rolled."*

In the course of our investigations I had occasion to make an examination of the material in the excavations, available for the inner lining of the embankment, and for surfacing the bottom of the reservoir preparatory to covering it with concrete lining, and find that at the south end near stations 59 and 68 there is a limited quantity of sandy material, containing 25 and 30% clay, respectively, in the two samples examined, and that the same might be made to answer the above purposes.

Also that in the excavation opposite station 24, of the aqueduct line, there is a considerable quantity of coarse gravel, of a suitable quality for mixing with the fine sand in the soft places in the bottom of the reservoir, in order to prepare it for the concrete lining.

There is a limited quantity only of the above materials, and it becomes a question, if the present plans are to be adhered to throughout the entire length of embankment, on the southerly and easterly sides of the reservoir, whether there will be a sufficient quantity of good material to properly prepare the slopes and bottom.

It would appear to me to be necessary therefore that a careful examination should be made of the material underlying the bottom of the reservoir where it is weak, or yielding, and of the material at the foot of the inner slope of the embankment in the depressions, where the core-wall is not founded upon the rock, and such further modifications be made in the plans as will beyond doubt make the reservoir safe, and its bottom and banks as nearly as possible impermeable to water.

It is not my intention to disturb in any way the conclusions arrived at by the Board of Engineers, on which I served, having been in entire accord with my colleagues, that it was not essential to the stability of the embankment, and the security of the reservoir against *dangerous* leaks, that the core-walls should in all cases be founded on the underlying rock.

There are technical reasons for this finding which are fully set forth in the report, and which need no further elucidation here. As to the correctness of the conclusions therein expressed, I think there can be no question.

It is admitted, however: (1) That there will be seepage through the botttom of the reservoir and the lining of the slope of the inner embankment and that the resultant outflow must be taken care of by the sewers in the adjacent avenues.

(2) That the making of the entire interior surface of the reservoir impermeable to water is dependent upon lining the slopes of the embankment with a paving of stone or brick laid in concrete, and its bottom with a layer of concrete, *on good earth*, compacted by rolling, as provided for in the plans and specifications.*

The specifications attached to the contract contain the following provisions in reference to the construction of the embankments and bottom of the reservoir:

Item 2.—Test pits and borings have been made to ascertain the nature of the ground where the work is to be built, and the results are shown on the plans. It will be noticed that the test pits and borings indicate, on the site of the reservoir, the presence under the superficial soil and underlying earth, of a large quantity of water bearing sand; leaving generally, between the superficial soil and the sand, a thin stratum (of varying thickness) of earth suitable for the making of the reservoir and other embankments. Should the character, location and extent of the various materials be found to differ from what is indicated by the test pits and borings, the contractor shall have no claim on that account, and it is expressly understood that the corporation of the City of New York does not warrant the indications of the tests to be correct.

Item 3.—All work must be built in accordance with the plans and directions which shall be given by him (the engineer) from time to time, *subject to such modifications and additions* as said engineer shall deem necessary, during the prosecution of the work.

Item 5.—The test pits show that water stands near the surface, especially on the reservoir site. The contractor is to do all the pumping and draining necessary for keeping the work free from water.

Item 26.—The surfaces which are to receive embankment must be stepped if so ordered. All earth bottom of the reservoir *must be rolled and watered* before the concrete is laid upon it, the weight of the roller and the frequency of the rolling is to be approved by the engineer. The cost of such rolling is to be included in the price stipulated for the earth excavation.

Item 44.—To compensate the contractor for the extra cost of the careful blasting close to the reservoir bottom and slopes, and for the preparation of the rock surface which is to receive the concrete—a price, clause O, item (d), is to be paid for each square (100 sq. ft.) of the said rock surface thus prepared.

Item 51.—The cost of doing all the work of refilling and embankments is included in the prices herein stipulated for earth and rock excavation.

Item 52.—The embankments for the reservoir shall start from a well-prepared base, stepped if so ordered on sloping ground; all embankments and all refilling shall be carried up in horizontal layers, not exceeding 6 ins. in thickness; every layer to be carefully rolled with a heavy grooved roller, and to be well watered. The earth to be well rammed with heavy rammers at such points as can not be reached by the roller.

Item 57.—The engineer shall decide upon the quality and character of the materials to be used at various places, and they must be selected and placed in accordance with his orders; the most compact earth must be used on the inner side of the center wall of the reservoir embankments.

Item 58.—When the engineer finds it necessary to mix the materials to be used for making embankments and for refilling, separate loads of the various materials designated by the engineer and in the proportions to be determined by him, shall be deposited on the work at proper intervals; they will then be thrown with shovels or otherwise in such manner as to effect a thorough mixture.

Referring to Item 51 of the specifications: It is not in accord with the best engineering practice that no provision should be made for paying an extra price over and above the cost of common excavation, for the making of the lining of the inner slope and the bottom of a reservoir. This part of the work requires great care. It must be made of carefully selected material, watered and rolled and as nearly as possible impermeable to water. This is essential whether there is a concrete lining or not, and the want of attention to this detail has been the cause of many failures in reservoir construction.

From examinations which I have made of the material available on the site, I am of the opinion that the greater part of it is not of good quality for the lining of either the banks or the bottom of the reservoir. It is true that the latter will be protected against leakage by a covering of Portland cement concrete of 6, 8 or 10 ins. thickness, at the discretion of the engineer, and that the slopes are likewise protected by stone and brick paving, laid on Portland cement concrete, but nevertheless the preparation of the slopes and bottom for the reception of the concrete will require great care.

There are considerable areas of bottom now excavated to grade, which are underlaid with a stratum of fine sand, some of it minutely divided, which is not stable when saturated with water. This is its natural condition and the bottom will, in my opinion, require extra preparation to fit it for receiving the concrete lining. An examination of a fair sample of this sand shows that it contains no clay, nor binding material whatever. Some of the thin strata contain a large percentage of disintegrated mica, causing it, when agitated with water, to behave like quicksand, although it is not a true quicksand, and when *drained* of water it is stable.

This material is objectionable on the bottom of the reservoir as a base for the reception of the concrete lining because of the danger of the concrete cracking and forming fissures through which water may find its way, to still further saturate the underlying sand when subjected to the pressure of from 24 to 26 ft. of water in the reservoir.

Whilst this would not disturb the core-wall, for the reasons stated in the report, with its foundation resting on sand at elevation + 98.0, which is 10 ft. below the bottom of the reservoir, yet it would no doubt lead to claims for damages against the city on the part of owners of property on the adjacent avenues after the reservoir is put into service.

This is not an unusual sequence of reservoir construction, and in this case it would matter not whether the water came directly from the reservoir or from the saturated sand strata underneath. It would nevertheless be attributed to the reservoir. It is altogether probable too that large buildings will in time be erected on Kingsbridge Road and Jerome Ave., opposite the reservoir, with deep basements, rendering them particularly liable to infiltration of water from the surrounding and underlying sand strata, the damage from which would be attributed to a leaking reservoir.

It is this feature of the case that makes it not only desirable, but necessary, that every known precaution should be taken during the construction, to make the reservoir as nearly as possible water tight. Its effectiveness to resist seepage and leakage will depend not only upon the stability and impermeability of the core-wall and embankment, but also of the bottom.

There have been in the past numerous illustrations scattered over the country of defective reservoirs, and many cases of absolute failures, requiring the expenditure of large sums of money to correct mistakes, which might more easily, and at less cost, have been avoided in the original construction. I need refer to only two examples:

The Queens Lane Reservoir, a distributing basin of the City of Philadelphia water supply, which I have selected because the rock underlying the site, like that at Jerome Park, is gneiss and mica schist, with a nearly continuous covering of sandy clay. The inner slopes of the reservoir were originally covered with concrete, diminishing in thickness from 12 ins. at foot to 6 ins. at top. On the bottom it was covered with 4 ins. of concrete which overlapped the foot of the slope lining. The concrete lining, bottom and slopes, was laid upon a layer of clay puddling 2 ft. in thickness, the material being well selected, built up in layers, rolled and watered. This is heavier lining than has been provided for in the specifications for Jerome Park Reservoir. When filled to a depth of 10 ft. the reservoir leaked so badly that it was found to be necessary to draw the water and entirely reconstruct the bottom and slope linings. Under a head of 10 ft. the north basin leaked per 24 hours, 0.70 ft. depth, and the south basin 1.25 ft. Under a head of 4 ft. the north basin leaked 0.35 ft. depth, and the south basin also 0.35 ft. Upon careful examination after the water had been drawn, the concrete lining of the bottom and inner slopes was found to be seriously disturbed and broken, admitting water to the underlying material and causing extensive leaks, which showed in running water on adjacent properties. The cost of reconstructing the interior lining of the reservoir, which is of moderate dimensions as compared with Jerome Park, was nearly $275,000.

The Roxborough Reservoir, also of the City of Philadelphia water supply, and of somewhat similar construction as Queens Lane, proved leaky from the same causes, and an expenditure of nearly $140,000 was required to reconstruct the interior lining.

In both cases it was found necessary to resort to asphalt treatment to make the sides and bottom tight.

These are only two examples of defective construction of reservoirs recently built under plans and specifications carefully prepared, and which were supposed by their authors to be all sufficient. Many others could be cited, for there are innumerable examples which only serve to emphasize the fact, that plans and specifications, however well drawn, do not build a reservoir.

It is admitted that such a condition as an *absolutely water-tight* reservoir or earthen dam does not exist, but all such basins, especially when situated in close proximity to the built-up districts of a town or city, should be as nearly water tight as possible.

In order to guard against any outflow of water resulting from seepage and possible leakage from Jerome Park Reservoir, it would appear from the existing conditions that one of two things must be done, namely:

Either completely surround the two basins with retaining walls or core-walls, laid in Portland cement and carried down to a solid rock foundation, confining not only the water but the material of the embankments where they exist, and the underlying sands of the bottom as well; or make such changes in the make-up of the inner slope of the embankment, the preparation of the bottom, and the concrete linings, as will make them as nearly as possible impermeable to water.

As to the first proposition; the treatment suggested has practically been adopted for the entire westernmost basin between the aqueduct line and Sedgwick Ave., except from Gate House No. 2 to the old Croton Aqueduct, which is in embankment, with a masonry core-wall carried down to solid rock. There appear to be good and sufficient reasons for this change from the original plans, namely, the sub-

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*(We have looked for such admissions as these in the report of the Board of Engineers. On (1) we find the following:

"The utmost that can be anticipated is the seepage of a small amount of water through the embankment and the earth, and this would be carried off by the sewers in the adjacent avenues. A large expenditure to absolutely prevent such seepage would not be warranted nor advisable."

On (2) we find no admission, whatever, but merely a statement of what the "plans and specifications provide," as follows:

"The plans and specifications provide for lining the interior slopes of the embankment with a paving of stone or brick laid in concrete and its bottom with a layer of concrete, on good earth compacted by rolling, with the intention of making the entire interior surface of the reservoir impermeable to water. They further provide for the construction of cement masonry core-walls in all the reservoir embankments."—Ed.)

stitution of retaining walls for inner embankments. The bottom rock is at a high elevation, making the construction comparatively easy, and having the advantage of greater permanence than earth work. Taking into account the scarcity of good material for embankment, the change made is certainly in the line of a betterment of that section of the reservoir.

Nothing will remain to be done, therefore, in this section, but to properly prepare the bottom for the reception of the concrete lining. It will be surrounded when completed with masonry, either core-wall or retaining wall, founded on rock, and should be as nearly as possible water tight.

In the easternmost basin, along Kingsbridge Road and Jerome Ave., the case is entirely different. At stations 74 + 90 to 77 + 60, a distance of 270 ft., a retaining wall would be required with a maximum height of 67 ft., taking the place of a masonry core-wall, whose foundations now extend in fine sand to a depth 10 ft. below the bottom of the reservoir.

At stations 84 + 00 to 90 + 80, a distance of 680 ft., a retaining wall with a maximum height of 56 ft., would take the place of a masonry core-wall, whose foundations now extend in fine sand to a depth of 10 ft. below the bottom of the reservoir.

At stations 97 + 30 to 99 + 80, a distance of 250 ft. along Jerome Ave., a retaining wall would be required with a maximum height of 68 ft., to take the place of a masonry core-wall laid under conditions similar to the above.

The only practicable location for such a retaining wall would be at or about in line with the foot of the slope of the embankment at each location, with return ends, to join with the core-wall now founded on rock. This plan would involve the construction of not less than 42,000 cu. yds. of rubble masonry laid in Portland cement, costing approximately $175,000.

The alternative is to make the best use of the material available, and so construct the inner slope of the embankment on the southerly and easterly sides, and the bottom where it is soft, as to reduce the seepage to a minimum.

In the section bounded by Kingsbridge Road and Jerome Ave., the inner embankment, on which no work has been done for several years, is made, as far as it has been constructed, of light sandy material, run in by scrapers from the adjoining soft bottom. It will require an average thickness of 4 ft. of lining to properly prepare the inner slope, from Station 74 (Gate House No. 6) to Station 104, and for this purpose about 37,000 cu. yds. of good material will be required.

This can be obtained from the earth overlying the rock, between Stations 42 to 46 and 300 to 600 (ft.?) east, where there is a depth of 8 to 10 ft. of suitable material, amounting to approximately 35,000 cu. yds. The deficiency can be made up from a small deposit of good earth in the neighborhood of Station 60, 200 (ft.?) east, containing a few thousand cubic yards. These two remaining deposits embrace all the good puddle material available on the reservoir site. It contains 25 to 30% of clay, mixed with sharp sand, and little or no mica. I would recommend that it be carefully saved for embankment work.

In the treatment of the line of embankment between Stations 74 and 104, I would recommend that footing walls be built at the toe of the slope, crossing the three depressions where the core-wall is not founded upon rock, the depth of the wall to be left to the judgment of the engineer, but I would consider a depth of 8 to 9 ft., making the foundation + 99 or 100, as sufficient. These walls will answer the double purpose of supporting the embankment on soft material, and to act as an additional cut-off, for seepage water moving in the direction of the core-wall. In the reconstruction of the Queens Lane Reservoir, before referred to, these walls were carried down to solid rock, which does not appear to be feasible in the case of Jerome Park Reservoir, except at great cost.

I have already called attention to the lack of good material in the western section of the reservoir. There is a large area of bottom extending from Station 27 to 30 + 60 and irregularly in shape from 3 + 50 to 10 + 70 west, measuring in area as now developed, about 3.8 acres. It will increase as additional bottom is put down to grade. This is boggy and miry, and requires special treatment before laying the concrete floor. I would recommend the use of coarse sand and gravel from the bank in the neighborhood of Stations 25 to 36 and 400 to 700 (ft.?) east. This is excellent material for mixing with the fine water-bearing sands and thereby consolidating them and strengthening the bottom.

The same conditions exist on the east side from Stations 12 to 34 and irregularly from 100 to 600 (ft.?) east, about five acres in extent. The bottom is boggy and miry and requires special treatment with gravel and coarse sand, as indicated above.

Also on the east side in Section N, from Station 66 northward to Station 49 and irregularly from 100 to 600 (ft.?) east, an area of 5.8 to 6 acres of bad bottom exists which requires special treatment. It was from this section that the soft top soil was taken by scrapers and made into embankment. The bottom should be excavated sufficiently below grade to permit of being filled up with better material.

The specifications attached to the contract allow sufficient discretion on the part of the engineer in the matters of selection of material, stepping the slopes of the embankments, rolling and wetting down the material, the preparation of the bottom for the reception of concrete, and the thickness of concrete to be used, to insure a well-built work, and I have only to add that with the exception of the inner lining as far as constructed in Section N, all the work of the contractor appears to have been well done.

## FURTHER DETAILS OF THE FAILURE OF THE PORTMAN SHOALS DAM, NEAR ANDERSON, S. C.

In our issue of Jan. 9, we gave a brief account, accompanied by a cross-section, of the failure of the dam of the Anderson Water, Light & Power Co. Through the kindness of Mr. Geo. H. Crafts, an engineer of Atlanta, Ga., we are enabled to present four views of the wrecked dam, taken on Jan. 1. The dam was built across the Seneca River, some ten miles from Anderson, S. C. It failed about midnight on Sunday, Dec. 29. The accompanying views indicate that the damage extended further down into the original dam (see below) than was reported in our issue of Jan. 9.

Figs. 1 and 2 are views of the wrecked dam from below, and Fig. 3 shows one end of the portion left intact. Figs. 2 and 3 indicate quite clearly the line between the old dam and the addition made in 1901. Fig. 4 shows some bad cavities in the masonry, while in Fig. 2 may be seen a large mass of masonry that held together and was carried down stream. We quote from Mr. Craft's letter to this journal, dated Jan. 10, as follows:

The old dam was built in 1895 (or 1897—Ed.) and was about 23 ft. high, 21 ft. wide at the base and 8 ft. at the top. Last summer an addition was made by building a section about 11 ft. wide outside of the old dam (making in all 32 ft. width at the base) and with 8 ft. top width as before, increasing the height 21 ft., making it 44 ft. high in all. Apparently there was no junction between the old and new dams below the top of the old dam, but immediately at the top of the old dam railway rails were used to tie the two together.

The spillway was 421 ft. long and the water was about 7½ ft. deep over the top of it when the dam burst, which was a tremendous flow. The dam had been practically tested previous to this flood, having had quite a depth of water flowing over the spillway and no weakness showing where it broke. The old dam was built by W[illegible] & Hardaway, and the new portion by Geo. W. Tenn[illegible]

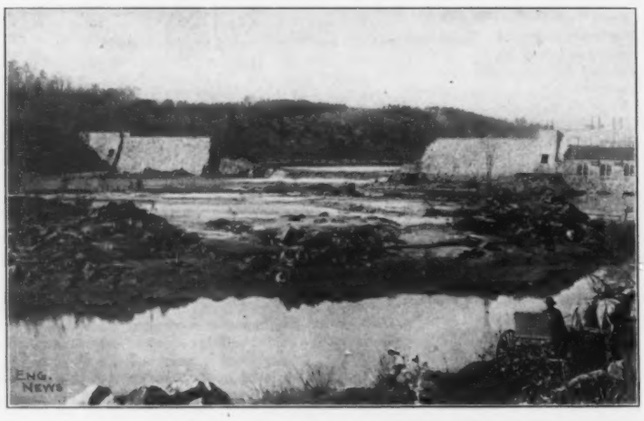


Fig. 1. General View from Below.

Fig. 2. Nearer View from Below.

VIEWS OF WRECKED DAM AT PORTMAN SHOALS, NEAR ANDERSON, S. C.

## ANNUAL CONVENTION OF THE MICHIGAN ENGINEERING SOCIETY.

The 23d annual convention of this society was held at Grand Rapids, Mich., Jan. 7, 8 and 9, the sessions being held in the council chamber of the City Hall. About 40 members were present.

At the first meeting on Jan. 7, an address of welcome was delivered by the Mayor, Mr. Perry, and this was responded to by Mr. Wm. B. Sears, President of the society. Mr. E. W. Muenscher then read the report of the Committee on Topographical Survey of the State, showing that a memorial had been presented to the Legislature asking for an appropriation of $40,000 for this purpose, and that the U. S. Geological Survey would provide an equal amount this would give $80,000. No appropriation was made, but the matter will be again brought up at the next session of the Legislature. In the meantime, the State Geologist, Mr. Lane, has set aside $2,000 for the work from his own appropriation.

By request of the Mayor, Dr. E. W. Bessey then brought up the question of the city's water supply, which at present is very bad. He advocated a supply from artesian wells, if a sufficient quantity could be obtained in this way; and next to this he recommended Grand River. Mr. Muenscher considered that the river was the natural source of supply, and that neither private firms nor the government should be allowed to continue the present pollution of its water by sewage. Mr. Bacon, of New York, described the Bacon air-lift system of pumping from deep wells, and thought a sufficient supply could be developed in this way, as has been done at Jackson, Mich.

Dr. Bessey made the common mistake of appealing to the society for assistance in the city's difficulty, for it is certainly not the function of engineering societies to do work for nothing which should properly be entrusted to experts, who have made the particular class of work a specialty, and who should be specially engaged and paid by the city.

At the evening session the President, Mr. Wm. B. Sears, delivered the annual address, and a paper on "Drainage, Heating and Ventilation," by Mr. A. B. Raymond, of Detroit, was presented.

At the meeting on Wednesday morning, the first paper presented was by C. Y. Dixon, of Amherstburg, Ont., on "Methods of Making Hydrographic Surveys and Carrying on Government Improvements." This described methods of laying out work on the ice, awarding contracts and the measurements of excavation.

A paper on "Backwater," by R. E. Horton, of Utica, N. Y., was read in the absence of the author and proved very interesting, developing some discussion. Mr. E. W. Muenscher, of Manistee, thought the subject was of importance, but doubted if any but approximate results could ever be obtained by mathematical methods. Messrs. Hodgman, Strong and Williams referred to individual cases, particularly in regard to the relation of ice levels to the actual levels of backwater. This was followed by a paper on "The Dowagiac River Improvement," by W. H. Goss, of Bangor, Mich. An abstract of this is given below:

### IMPROVEMENT OF THE DOWAGIAC RIVER FOR DRAINAGE PURPOSES.

This Michigan river is a tributary of the St. Joseph River, and is about 25 miles long on an air line, or at least three times as long if the river course is followed. About two-thirds of the length, at the upper end, is through a swamp 1 to 2 miles wide, and the river is to be improved to drain this swamp, as the land is very fertile

staking out the drain or new and improved channel, grade stakes were set in the center line two chains apart, and a hub stake was set at each grade stake that was not in the river. A 66-ft. chain length was used in measuring the distances, as that put them on the same basis as the land lines, thus making the computations easier. No slope stakes were set, as the notes give the width at each grade stake, and this width can be easily determined from the center stake. A railway compass was used in running the lines, as being more portable than a transit. The grade stakes were marked with arabic numerals and the angle stakes with roman numerals, cut with a timber scribe. Plans were also made for connecting the different courses by curves of 2 chains radius.

In the field work, the angles were first located and stakes set, and then the grade stakes were set in line between those at the proper distances apart. A boat was used for setting the stakes at deep holes. For measuring, a No. 14 hard brass wire 2 chains long was used, which was very light and convenient, and remained of constant length, as found by frequently testing it with a steel tape which was used to measure the fractional distances between the grade stakes and the nearest angle stake. When the survey lines were completed, leveling was commenced at the lower end of the drain. Benches were formed by nails driven in trees used as witnesses to the angles.

The length of the drain is 1,002 chains 75 links, with a difference in elevation of 34.6 ft. The lower part has a grade of 2 ft. per mile, and the upper part 2.8 ft. The average depth of drain is 5 ft. in cuts, and from 0 to 3 ft. in the river channel. The area to be drained is 15,000 acres, and the bottom width of drain is 20 ft. in the lower portion and 16 ft. in the upper portion. The total length is 12½ miles, on an air line of 11 miles, and nearly half of it is new channel. There are no heavy cuts, and no indications of quicksand, but there appears to be a stratum of gravel about 5 ft. below the surface. The work will necessarily be done with a dredge. This drain is one of several similar ones in the same section of the State, and while each will cost considerable money, they will cost less per acre of land benefited than many smaller drains.

In the discussion some members thought it would have been better to use 100-ft. stations instead of 66-ft. chains, regardless of land lines, but Wr. F. Hodgman, of Climax, thought that in such cases as this it was wise to adopt the chain as the land lines had been laid out in the same way.

Fig. 3. View Showing Cross-Section of Wrecked Dam Near Anderson, S. C., and Junction of Old and New Portions.

Mr. Goss remarked that many local contractors do not understand measurements in tenths of a foot, and he therefore reduces his levels to inches.

A paper on "The Corona Water System," by A. L. Reed, of Whittier, Cal., was then read, describing a water supply for irrigating 3,500 acres of orange groves. It includes cement and wood-stave pipe and cement-lined conduits. This was followed by a paper by W. J. Sherman, of Toledo, O., on "Electric Railway Construction." The paper dealt mainly with the general development of electric traction for city, suburban and interurban lines, and called particular attention to the enormous losses between the boiler furnace and the car axle. The author also suggested greater care to provide for operating expenses and fixed charges in less prosperous days and with plants that will then have deteriorated. In the discussion Mr. E. W. Muenscher, of Manistee, referred to the proposition to establish generating stations at the coal mines, in order to reduce the present losses.

In the afternoon the party visited a number of the city's public works, under the guidance of Mr. Anderson, City Engineer, and Mr. B. Meyer, superintendent of the electric light and water-works. The water-works pumping plant was first visited, which includes a 10,000,000-gallon triple-expansion horizontal Nordberg engine, and a 15,000,000-gallon Holley engine for high-pressure service. A new concrete-steel arch over the power canal was inspected, and then the street railway power station, which will eventually develop 4,500 HP. The Engle garbage crematory and the city electric lighting station were then visited, the latter having high-speed engines with steam slide valves (main and cut-off) and Corliss exhaust valves, built by the Russell Engine Co., of Massillon, O. The party also visited the Pere Marquette R. R. bridge over Grand River, where concrete piers are being built for a new superstructure. The stone is heated and hot water is used for the concrete.

In the evening there was a business session at which the reports of the Secretary, etc., for the fiscal year ending with September, 1901, were presented. Nominations were made for officers for the next year, and these will be balloted for next September. For the place of the next meeting, Battle Creek was suggested by Mr. Brigden and endorsed by Mr. Hodgman.

Mr. A. L. Holmes, of Grand Rapids, then read a paper on "Water Cost and Water Waste," showing the great expense which cities incur in supplying water without any check upon the consumption, and the saving which may be made by the general use of the meter system.

On Thursday morning, Prof. J. B. Davis, of the University of Michigan, gave a most interesting address upon "The Survey of the St. Clair Flats," upon which work he has been engaged as Chief Engineer, conducting the work for the State, with Mr. Fred. Morley as Resident Engineer. An abstract of his address is given below:

## SURVEY OF THE ST. CLAIR FLATS.

These flats form a delta at the mouth of the St. Clair River, and vary from 3 ft. above to 6 ft. below the water. They are outside the territory of the Land Survey, but parties gradually settled on them and established hotels, summer residences, etc., until eventually they began to interfere with each other, while some attempts were also made to enclose land. The residents have denied the jurisdiction of the lower courts and have in several cases refused the payment of the road and school taxes, as there are no roads or schools within the territory. In view of these facts, the State Legislature passed an act assuming control of the lands, and undertook to have them surveyed and the titles settled. All existing channels navigable even by duck boats (about 8 ins. draft) are to be retained and made free of use, and lots were to be laid off fronting on Lake St. Clair and not exceeding 500 ft. in depth. The property between this limit and the limits of the Land Office survey is set aside as a public park.

As the object of the survey was to settle titles (which so far had no basis) and not to disturb them, the people were first interviewed by the engineer, their stories as to boundaries, claims and improvements were heard and recorded, and matters were so conducted that they felt that their interests would be looked after fairly. There were found some 600 lots of every form and size, with frontages of 30 ft. to half a mile. For these there were 400 claimants, but only four disputed claims have arisen, which is probably due to the above method of interviewing and informing the people first, instead of making an arbitrary survey in the first place. The office was on a boat, so that the people could readily come and state their cases.

For monuments, 1½-in. wrought-iron pipe was adopted, and all are numbered by means of a center punch, making the figures with rows of dots. To support the transits over these, light pipe trestles were made, having three posts and two rows of girths, the posts being capped for driving. The tripod legs rest in adjustable cups sliding on the posts. The transitman occupied a platform carried by two scows 12 × 4 ft. 14 ins. deep, and in the center of this platform (between the scows) was a hole 4 ft. square for the pipe trestle. At each corner of the platform was an upright 4 × 4 ins., 3½ ft. high, outside of which was an oak spud, 4 × 4 ins., notched at the top to receive a transverse plank, held in place by pins. Each plank was connected to the platform by a tackle. At each corner was also an anchor. The four anchors were sufficient to get and hold the transit scow in exact position, but the ripple of the water moves the scow enough to prevent a man from keeping his eye at the telescope. Consequently the spuds are dropped, and the tackles hauled in sufficiently to raise the craft slightly (not bringing the scows out of the water) and so give a steady working platform.

The monuments were first driven with mauls, but this was very soon found to be unsatisfactory, and was, of course, utterly impracticable in water, the pipes being driven down 4 or 5 ft. below the water, so as not to interfere with navigation. Working with a follower broomed the pipes badly, until a steel cap and follower were devised. This apparatus when perfected was so heavy that the pipe was driven by simply "churning" with the follower, which had two side handles to be adjusted at any required height. As plumb bobs could not be used, a "plumb staff" was devised, consisting of a 1¼-in. pipe fitted with a steel point of convex form to enter the top of the monument pipe. This was plumbed with a machinist's level and gave very accurate results.

A 500-ft. steel tape was used for measuring distances, as all stakes on the overflowed lands would have to be pipes driven as described. This was supported on the water by fish-net floats, and the front end was held by a man on a scow similar to the transit scow except that it had no poles or spuds. The principal work was done with a 4-in. Buff & Berger transit, so small that it could be easily carried.

Some of the channels are 30 ft. deep, except at the ends, where there is only about 18 ins. of water. These are scooped out by the ice blocks which pack almost solid to the bottom of the channel, but when the ice melts it does so from the top and the water passes out over the shallow end. The ice is very treacherous, it not only lets men fall through when it is apparently solid, but it will suddenly and unexpectedly develop a slight creeping or movement which will undo a lot of work done in staking out, so that it became necessary to drive each stake permanently as soon as it was set. Men moving about in groups were required to hold on to a rope. When a man did fall in, as soon as he was got out he was rushed at once to the house boat, put in a warm room, stripped and rubbed, and given food if he asked for it. Under such treatment the man was generally able to return to work in a short time. The maps show all buildings, fences, sheet piling, filled pieces of land, ditches, drains, trees, walks, and all improvements. Also all monuments, lines, main traverse lines, etc. The direction and length of every line is given. The scale is 80 ft. to the inch, so that measurements can readily be made with a pocket rule, and the size of the sheets is such that they will fold to fit the size of the deed. Every owner is given a blue print of the sheet showing his property. The maps were first made on drawing paper, and then cloth tracings made for office use and for blue printing, the original maps being bound and filed in the vault at the State Land Office for permanent record. All notes, etc., were copied, verified and compared by different persons to ensure accuracy, and a general map was made from the sectional maps, and all errors (which were very slight), thus found were rectified.

Mr. E. Strong, of Kalamazoo, then read a paper on "The Use of the True Meridian in Resurveys," which was followed by a paper on "The New U. S. Instructions to Surveyors, 1902," by A. W. Barber, of Washington, D. C. On motion of Mr. E. W. Muenscher, the following resolutions were then adopted:

That the Census Office of the United States be requested to give careful consideration to municipal statistics before closing its enquiry; and

That the society desires to express its opinion that it is advisable to establish a permanent census office, and that such permanent office should undertake the collection and publication of municipal statistics at intervals of less than ten years.

At the afternoon session Mr. H. M. Gifford, of Toledo, O., read a paper on "The Use of Concrete in Sewer Construction," describing a system of construction employing arch blocks, which we shall describe in a later issue. Prof. M. E. Cooley described tests which he had made on the concrete blocks, showing a tensile strength of 4,280 to 4,530 lbs., and a compressive strength averaging 1,049 to 1,080 lbs. per sq. in. He also mentioned a concrete conduit at Jerusalem, probably 2,000 years old, which has concrete blocks about 2 ft. long, 24 ins. outside diameter and 8 to 10 ins. inside diameter.

An interesting talk on "Boiler Explosions" was then given by Professor Cooley, which he prefaced by saying that he had expected to present the results of his investigations into the recent explosion at the Penberthy Injector Works at Detroit, but as the legal proceedings have been delayed by the illness of the engineman, who was injured, he was not at liberty to divulge these results. He discussed the common lack of boiler inspection, and its dangers, and also presented very clearly the inherent defects of boilers built with lap-joints, or in such a way as to have defective circulation. The popular public idea that disastrous explosions are usually due to low water was disproved by showing how the water behaves in case of explosion, so that a very disastrous explosion is fully good evidence that there was plenty of water in the boiler. He thought that legislation should be passed requiring the inspection of boiler materials and boiler construction. A short but interesting discussion followed, after which Mr. F. F. Rogers, of Port Huron, read a paper on "Details of Cost in Macadam Road Building." We shall give an abstract of this later. The last paper was on "Timber Cruising," read by Mr. D. A. Brotherton, of Escanaba, which was followed by some discussion as to timber conditions and reforesting in Michigan. Two papers read by title were as follows: "City Water Supplies from Reservoirs Tributary to Limited Watersheds," by George L. Wells, and "A Survey in the Navajo Reservation," by A. W. Barber.

Mr. Bean described a new method of designating township and section lines, and resolutions of thanks were passed to the Mayor, City Engineer and others for courtesies extended. The convention then adjourned.

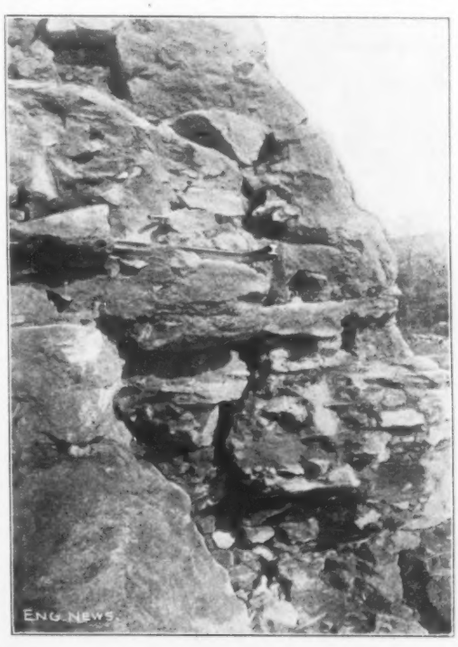


Fig. 4. Near View of Portion of Wrecked Dam, Showing Cavities in Masonry.

THE AMERICAN DEVELOPMENT CO., of New York, has been reorganized, and is credited with preparing to shortly commence construction on the Canton-Hankow Railway, for which it holds a concession from the Chinese government. This American line would be 750 miles long, and with projected branches it would be 900 miles long. At Hankow it would meet the Belgian Pao-Ting-Fu-Hankow line, of 650 miles, connecting with Pekin from the first-named place by a line started some time ago.

# ENGINEERING NEWS
AND
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*ADVERTISING RATES: 20 cents a line. "Want" and "For Sale" notices, special rates, see page XXIV-XXIX. New copy for standing advertisements should be received one week in advance of publication; new advertisements, Monday morning. The pages containing "Want," "For Sale" and "Proposal" advertisements are held open until Wednesday noon.*


A bill has been introduced in the New York legislature embodying Gov. Odell's recommendation that the proposed expenditure of $28,800,000 on improvement of the Erie Canal be submitted to popular vote. As noted last week, about half this sum is to be spent to build locks to accommodate 1,000-ton barges of 12 ft. draft—an expenditure which will be in large part wasted unless and until the taxpayers later consent to a burden of at least $75,000,000 more to complete a channed for such barges. Gov. Odell in his message compares his scheme for lock enlargement to the policy of a railway company, which strengthens its bridges to carry heavier locomotives before putting them in service. To make this comparison a parallel one we must suppose the bridges on the railway to represent a third to a quarter of the entire investment in permanent way, and that the company proceeds to rebuild these bridges to accommodate rolling stock double the present weight when it is wholly undecided that it will ever use such rolling stock or anything approaching it. We know of no railway company which was ever guilty of such absurd financiering.

Any such hybrid and half-way plan for canal improvement as that which Gov. Odell has proposed deserves, and is pretty certain to receive, final defeat.

If New York is to spend money on canal improvement, she ought to undertake the work on a scale that will at least not be obsolete before it is begun. If she can afford to spend $30,000,000 on a 9-ft. channel, as is now proposed, she can afford to spend $55,000,000, which is the engineer's estimate for a canal by which 1,000-ton barges can pass from Lake Erie to the Hudson River. Before undertaking such a work, however, she ought to test the question whether a 21-ft. channel, large enough to accommodate all the Lake shipping, cannot be built by the National Government. If the State of New York, before spending $28,000,000 or $55,000,000, or any other sum on a barge canal, will go before Congress with an offer to pay, say, $50,000,000, toward the cost of a 21-ft. deep waterway from the Lakes to the Hudson, there is strong reason to believe that the offer would be accepted, and that Congress would vote the remaining $155,000,000 estimated as the cost of such a route. Such a canal would probably carry double the traffic of a canal across the Central American isthmus, and is far more likely to justify itself as a commercial transportation route. If New York will take the lead in some such way as we have indicated, such a canal may be built; but so long as New York potters away with the obsolete old Erie Canal route, the National Government is not likely to move.

The Jerome Park Reservoir controversy has more than local interest or we should not again devote so large an amount of space to that subject as is taken up in our present issue. As will be seen from the report published elsewhere in this issue, Chief Engineer Hill comes out with a flat-footed non-concurrence with the report of the board of engineers (see Engineering News, Nov. 28, 1901). He even goes so far as to predict the destruction of the whole reservoir if the changes in the construction of portions of the embankment are not made. Mr. Smith, too, in a special report, which we also publish in full, while disclaiming any intention to disturb the conclusions of the board of engineers (of which he was a member), discusses in great detail the steps which he considers necessary to make the bottom and sides of the reservoir water tight. He appears to recommend masonry cut-off walls carried to rock, where rock is not too far below the surface, and special care with the lining of the inner slopes elsewhere. What he says about the bottom lining is foreign to the discussion heretofore raised.

The points at issue which are of most general interest are: (1) Is the probable amount of seepage through the Jerome Park Reservoirs, if built as originally designed, of sufficient moment to warrant changes in plan? and (2) if so, should the seepage be prevented by cut-off walls carried to rock, or by means of heavier embankments and greater care with the slope and bottom linings?

This raises the whole question of the imperviousness of core walls and foot walls, and the relative efficiency of either or both as compared with reservoir linings. We have no idea of attempting to answer these questions in the abstract, much less in their relation to the present controversy. But we have no hesitation in saying that the differences in opinion between the board of engineers and the Chief Engineer of the Aqueduct Commission, confused as they are by a special report from a single member of the board of engineers, demands that all the points now at issue be referred to a new board of engineers. This is all the more imperative since both the character of the work done by the contractor and his profits on the balance of the contract are at issue.

It now looks as though the people of Havana have a fair prospect of securing the early construction of the long and greatly needed sewerage and paving systems, and have also been saved the loss of money and self-respect which would have resulted from the award of the contract to Mr. Michael J. Dady, of Brooklyn. Mr. Dady's manoeuvres, political and otherwise, to secure this contract were partly reviewed in our issue of Jan. 3, 1901. Since then the government bought Mr. Dady's rights to the contract, gained under the Spanish regime, and threw the work open to competition. Mr. Dady and associates were among the high bidders and the contract was awarded, (or about to be awarded) to the lowest bidders, when Mr. Dady reduced his bid by $500,000, on the ground that he had based it on payment in bonds at 5% discount. This still left his bid about $220,000 above the lowest, but he won the hearts of the city council by promising to employ natives only on the work, whereupon it was reported that Mr. Dady would have the contract after all. But now comes what appears to be authentic information from Havana that the contract between the city and the lowest bidders, McGivney & Rokeby, of Jersey City, has been signed by representatives of both parties. The contract price is $10,649,000. This consummation, if the reports prove to be true, may be looked upon as another victory for our American officers in Cuba, and particularly for Governor Wood.

### THE PARK AVENUE TUNNEL DISASTER.

Seventeen persons dead and thirty-four seriously injured is the summary of casualties resulting from the rear collision in the Park Ave. tunnel in New York city on Jan. 8. The circumstances of the accident were briefly noted in our last week's issue, which went to press just after the first news of the accident was received; but in view of the importance of the wreck we deem it worth while to again and more fully present them.

The Park Ave. tunnel, in which the accident occurred, is about two miles in length and is located just north of the Grand Central station and yards in New York city. All passenger trains entering New York city from the New York Central and its controlled roads and the New York, New Haven & Hartford R. R., pass through this tunnel. There are four tracks in the tunnel, two of which are in a central tunnel and two in single-track tunnels on either side. Besides the regular incoming and outgoing trains, a large movement of empty-car trains through the tunnel is necessary as the yards at the Grand Central station are too limited in area to store cars in. Consequently all cars entering the station which are not to go out again at once or which require to stand for cleaning have to be hauled through the tunnel to the car yards north of the Harlem River.

The total train movement through the tunnel is said to be from 500 to 600 trains daily. Much of this is concentrated in the morning and evening "rush hours" on account of the heavy suburban traffic on all the roads using the tunnel, and frequently the trains follow each other through the tunnel as close as the block signal system permits.

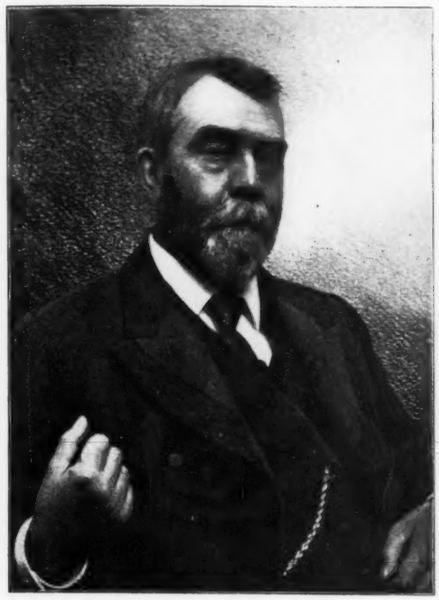
This block signal system is probably as perfect an installation as exists anywhere in the world. It is operated by towermen; but the instruments are so interlocked and so controlled by electric track circuits as to eliminate almost absolutely the possibility of a wrong signal being given. The signals are lamps turning on vertical spindles and are in duplicate on each side of the track. In the case of the track on which the collision occurred (and we presume on the other tracks as well) the signals on the fireman's side are at about the height of the cab. On the engineer's side the lamps are on low stands only a foot or two high. The colors used are red for danger, green for distant signals and white for safety.

Besides the visual signal, an automatic torpedo placing apparatus is connected to each home signal. This when the signal goes to danger shoves a torpedo in position to be struck and exploded by the wheels of any train which may run past the signal while at danger. There is also an automatic gong apparatus, which is arranged to sound at intervals whenever a train runs into a closed block.

Trains are run through the tunnel at speeds of 20 to 35 miles per hour. Head room in the tunnel is said to be so limited that the stacks of the largest passenger locomotives come within two or three inches of the tunnel roof, and thus the smoke and steam are deflected downward.

A typical cross-section of the tunnel was published in our issue of Oct. 17 last in connection with a description of the work of replacing the masonry partition walls which separated the side tunnels from the centre tunnel with steel columns and I-beams. The only ventilation of the side tunnels at present is through low arched openings in the tops of these masonry partition walls opening into the central tunnel. The change referred to consists in removing these walls and practically throwing the three tunnels into one.

On the northern portion of the tunnel there are few or no openings to the street. On most of the

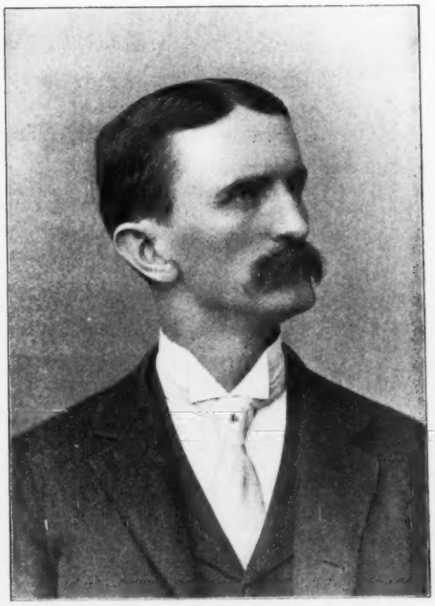


Robt. Moore

President of the American Society of Civil Engineers.

Edwin Reynolds

President of the American Society of Mechanical Engineers.



E. E. Olcott

President of the American Institute of Mining Engineers.

Chas. Proteus Steinmetz.

President of the American Institute of Electrical Engineers.

THE PRESIDENTS OF FOUR AMERICAN ENGINEERING SOCIETIES.

tunnel, however, the roof of the central tunnel is open for most of the distance between the cross streets, the opening being about 12 ft. in width and located over the center of the central tunnel.

Coming now to the events at the time of the accident, the trains in collision were a New York, New Haven & Hartford local from South Norwalk, Conn., due at the Grand Central station at 8.17 a. m., and a Harlem train from White Plains, due at 8.14. The Harlem train passed over the bridge near Mott Haven at 8.09. The New Haven train passed this point three minutes later, but was then switched onto the outside track and run past the Harlem train, which was held up by a Croton local train ahead. At 96th St. (the entrance to the tunnel) the New Haven train was switched onto the inner track, so that it entered the tunnel on the same track as and ahead of the Harlem train. The two trains were three minutes apart at 96th St., 2½ minutes at 86th St., and 3 minutes at 72d St. At 56th St. the New Haven train was held up by the block signal. It was then protected by a home signal at 59th St., about 475 ft. in the rear, and by a distant signal at 63d St., about 1,000 ft. further back, both of which were correctly set. Besides this, the rear brakeman of the New Haven train went back, probably not as far as the distant signal, with two red lanterns. He fixed a torpedo to the rail and then returned about 250 ft. toward his train. When the Harlem train passed him he threw his lanterns at the cab window.

Wisker, the engineer of the Harlem train, is a man of good habits, 42 years of age, and served as a locomotive fireman for nine years on the Harlem division before being promoted to an engineer's position last summer. He appears to have had his present run but a short time. Fyler, his fireman, was an experienced man, but the two were making their first trip together. Fyler, according to his published statement, was sitting on his side of the cab watching for signals as they passed through the tunnel. He saw the green light at 63d St. and called out "green" to the engineer on the opposite side of the cab. Then he saw the red light at 59th St., yelled "red" to his mate, and dived back into the tender.

The engineer, Wisker, is said to have shut off steam at 72d St., but he did not apply the brakes to check the speed of the train when he passed the green signal and is reported to have said that he did not see it at all. Whether he saw or heard the warnings at the home signal does not yet appear, but he evidently did not then apply the brakes or the speed of the train would have been materially reduced before it struck the train ahead. At some point between the danger signal and the point of collision he is said to have applied first the service and then the emergency; but the train was still running probably 17 to 20 miles an hour when the collision occurred. The Harlem train consisted of seven cars. The brakes were tested after the accident and found to operate properly. It is possible, however, that the pressure in the auxiliary reservoirs had been reduced by preceding applications of the brakes in the run from the Harlem River down, so that the pressures obtained in the emergency application were rather low.

The casualties in the disaster were confined almost wholly to the rear car of the New Haven train, into which the Harlem locomotive plowed its way until the smokebox was within 9 ft. of the front end of the car. Of the 60 passengers said to have been in this car, nearly all were either killed or received more or less serious injuries. The car next ahead also suffered somewhat, but all the other cars of this train were uninjured, except for broken glass, platforms, etc. On the Harlem train the shock appears to have been very slight. A passenger on the front platform of the car next the locomotive was not thrown off by the shock. So far as this train was concerned the force of the blow was absorbed by the crushing of the rear car of the New Haven train.

Undoubtedly the primary responsibility for the accident rests with the engineer, Wisker. He ran his train past both visual and audible warnings, and seems to have been as one paralyzed or in a trance for a few fateful seconds when the lives now sacrificed depended on the movement of his hand on the brake handle. It is easy to explain his failure to see the green and red lamps by the smoke and steam that commonly fills the tunnel, but the failure of the torpedo to arouse him to instant action would indicate that the shock of the sudden and imminent danger produced a mental paralysis that unnerved his arm for a few seconds just at the critical moment.

Official investigations are to be held by the Coroner and District Attorney, beginning on Jan. 16, and by the State Railroad Commission, beginning on Monday, Jan. 20, and we shall record later any further important facts bearing on the accident that may be brought to light.

The present accident is the third serious rear collision that has occurred in the tunnel. On Sept. 22, 1882, a suburban train from Port Chester was run into and telescoped by a New York Central train at 86th St., and some 20 persons were killed or seriously injured. The block system was not then in use in the tunnel. The second accident occurred on Feb. 20, 1891, when a New Haven local ran into the rear of a train of empty cars bound for the yards and shops above the Harlem. In the rear car of the train were a number of car cleaners and other employees, and six were killed and five injured. The investigation showed that the accident was primarily due to the smoke and steam which filled the tunnel and prevented the engineer from obtaining a clear view of the signals.

The present accident has aroused public opinion in a well-nigh unprecedented manner. The large number of casualties, the prominence of many of the victims, and the fact that many thousands of residents in the northern suburbs of New York city are daily exposed to the danger of just such a disaster as that of Jan. 8, all combine to cause a well-nigh unanimous verdict of condemnation against the railway management responsible for the maintenance of the tunnel in its present condition.

It is said that the engineer ought to have stopped if he was unable to positively identify the distant signal at 63d St., as the rules require; but it is a question which the official investigators will do well to sift, whether engineers do not frequently run past distant signals in the tunnel without positively determining their indication, or checking the speed of their train. In the murky atmosphere of the tunnel, fouled as it is by the passage of a locomotive every minute, it is frequently the case that only an expert can distinguish the signal lamps at all, and who can doubt that at times even the experts fail? It would be interesting to know how often it happens that engineers stop their engines to find out the significance of a signal which they fail to pick up. Are engineers permitted and encouraged to thus "err on the side of safety," or does the paramount ... cessity of getting the traffic through the tunnel make it necessary to "run chances" occasionally?

We do not know, and we doubt whether the official investigation will positively determine whether the engineer, Wisker, would have run his engine past distant and home signals as he did had he had a clear view of the track, unobstructed by smoke or steam; but this we do know, that the condition of the tunnel is and has been for twenty years a source of danger to the traveling public, as well as a nuisance to those who live beside it.

Eleven years ago a rear collision in the tunnel which cost six lives was shown to have been directly due to the obscuring of the signals by smoke and steam. The managers of the road have improved their system of signals, and have added automatic audible warnings to the other safeguards; but with strange and incomprehensible obstinacy they have refused to take a single step toward the removal of the smoke and gases which cause the tunnel to be such a source of danger to the public and a nuisance to those who live beside it.

We have frequently and forcibly urged that a system of ventilation should be adopted for this tunnel. There is nothing difficult or doubtful or experimental or even novel in such a system. It consists of nothing but such fans as are in common use for mine ventilation and other industrial purposes, engines or electric motors to drive them, shafts to discharge the foul vapors overhead, and proper inlets to the tunnel for fresh air. By such a system mines are ventilated with many miles of underground workings; and, strange though it may appear to the New York Central management, not a few existing railway tunnels have been and are being ventilated in this identical manner.

In the face of these plain facts, well known to the engineering profession, the company has not only made absolutely no effort to improve the condition of the tunnel by the installation of even the smallest and simplest ventilating apparatus, but has repeatedly informed the public that no such improvement was possible. Such a statement is not only absurdly false, but a libel on the engineering profession, and its ability to solve a very simple and ordinary problem.

The absurd obstinacy with which the company has refused to take a single step toward the ventilation of its tunnel is well illustrated by the changes it is now making in the tunnel partition walls. These changes will cost, we judge, as much or more as the installation of a ventilating system, and, when completed, the tunnel will be a less durable structure, less safe against a fall of the roof, and a greater nuisance to the dwellers beside it.

We are well aware that the public press has fixed almost unanimously on the adoption of electric motive power in the tunnel as the proper remedy for its condition, and it is currently reported that the company itself has decided to introduce it. For several months it has had Mr. B. J. Arnold, M. Am. Inst. Elec. E., at work upon plans for such a system; and it is said that announcement of the adoption of these plans is shortly to be made. It appears probable that these plans will provide for the hauling of all trains (except through express trains, and perhaps some or all of these as well) from yards above the Harlem to the Grand Central Station by electric locomotives. A stop for a change of locomotives at some point north of the Harlem will, of course, be necessary, and to arrange for this and for the network of third rail in the yards at 42d St. is the most difficult part of the problem. The difficulties are not insuperable, however, and whatever the intentions of the railroad company may have been before with reference to the electric system, the public sentiment which the accident has aroused are quit likely to fix the decision in favor of electricity.

Should this be the final result, it will be a fitting punishment for the corporation for its long years of neglect to properly deal with the tunnel problem. An electric locomotive system will cost, probably, forty times as much as a system of ventilation. Had the company been wise enough and progressive enough to have installed a ventilating system a dozen years ago, it might have prevented both the accidents of 1891 and 1902, and have saved to its stockholders the heavy expense of installing an electric motive power system.

We still maintain, as we have in the past, that the installation of electric traction for the tunnel is unnecessary, and that the necessary delays for changing locomotives which it will involve make it of doubtful benefit to the traveling public.

By placing large exhaust fans beside the tunnel, of a capacity to change the entire air in the tunnel every two minutes, or every minute if need be, it can be made as safe for the movement of trains, and as comfortable for passengers as the open road, and no greater nuisance to those living alongside the tunnel than it would be if electric motive power were used.

## LETTERS TO THE EDITOR.

### End-Area vs. Prismoidal Formula.

Sir: I note in your issue of Jan. 9 a letter by Mr. B. W. Hicks, entitled "A Peculiar Prismoidal Section," which Mr. Hicks seems to think shows the folly of depending upon the average end-area formula. Sections like the one given are very common in repair work on the Erie Canal, but I have yet to find an engineer who would solve the problem by using the upper and lower parallelograms as his "ends."

Railroad and canal engineers always use "ends" that are vertical, and not horizontal; and approaching the problem in the usual way we have at Station 0 a section that has A (area) = 0; at Sta. 0 + 45, A = 1,710; at Sta. 0 + 75, A = 1,710; and at Sta. 1 + 20, A = 0. Applying the end-areas formula and dropping fractions we have cubic yards

— 4,750. Turning to my formula given under "Rapid Earthwork Calculation," Engineering News, Dec. 13, 1900, and applying the correction to the four pyramids at the corners of the prismoid under consideration, we find by just one minute extra work, the exact correction is —750 cu. yds.; making the exact volume 4,000 cu. yds. We thus find, contrary to Mr. Hick's supposition, that the mean end-area formula gives an excess when properly applied and not a deficiency; and that there is available a simple and rapid means of correcting the mean ends formula so as to secure an accuracy equal to the prismoid formula, yet involving far less work.

Yours truly, Halbert Powers Gillette.

Rochester, N. Y., Jan. 10, 1902.

(Another correspondent, whose letter we do not reprint, misunderstood the problem stated by Mr. Hicks, and as others may have done the same, we think it worth while to restate the problem as follows: The prismoid to be computed is 30 ft. in length, between two parallel cross-sections, and (calling one of its dimensions width and the other height to distinguish them) it is 120 ft. wide at one end and 30 ft. wide at the other, while it is only 12 ft. high at the first-named end and 102 ft. high at the other. Thus, the midsection is larger in area than either end section, and the ordinary mode of computation by mean end areas is largely in error. Those who use it will appreciate from the above illustration the necessity of using also the correction method set forth by Mr. Gillette.—Ed.)

### A Diagram for Computing Flat Roof Beams.

Sir: In a flat roof building, the length of a sloping beam must be calculated along the line of slope and not on the horizontal. This will necessitate the calculation of the hypothenuse of a triangle, one side of which is the distance center to center of columns or beams, and the other side the rise of the beam. The accompanying diagram gives the increased length of the hypothenuse over the base for spans up to 50 ft. and rise up to 45 ins. I

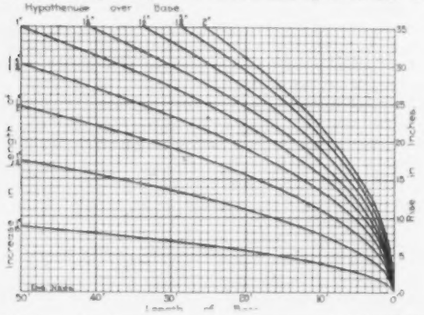



Diagram for Computing Hypothenuse of Triangles of Small Rise.

plotted the diagram, while having to check sloping beams of a flat roof building and found it to have an advantage for this purpose over Buchanan's or Hall's "Tables of Squares." I send it to you thinking it may interest some of the readers of Engineering News. Yours truly,

J. A. Ray, C. E.,
With the Dominion Bridge Co., Ltd.

Lachine Locks, P. Q., Dec. 15, 1901.

(The original diagram sent by our correspondent is plotted on cross-section paper with vertical lines at each 6-in. interval in the base, and horizontal lines for each half inch interval of height, while the curves are plotted for each 1-16 in. and the diagram may be read to 1-32 in. by interpolation. As the instances appear to us exceptional where such a diagram would have an advantage over tables of squares we show only enough of the diagram to indicate the principles on which it is made.—Ed.)

### Steel Bonds for Masonry Dams.

Sir: I have noted with considerable interest the communication in your issue of Jan. 2 from your correspondent, Mr. Gustav Lindenthal, in regard to the use of steel bonds in high masonry dams. In your issue of Dec. 19, 1901, and on the supplement sheet, are the plans for a steel-concrete chimney for the Central Lard Co., of Jersey City. In the foundation are twisted steel bars, crossed and recrossed, and located just above the pile heads, so that in the event of any unequal settlement the strains would be communicated through the twisted steel bars to other parts of the foundation, and so prevent serious damage to the whole structure.

There seems to be a very close analogy between this case and that of a high masonry dam, the pressure of the water against the dam corresponding to the pressure of the chimney on its twisted steel-concrete foundation. It would seem reasonable to suppose that what was considered an important safeguard or factor of safety in one case would prove equally as important in an analogous case.

During the recent floods in this section of the country, four large masonry dams were destroyed (or badly damaged.—Ed.) at the following places: One at Columbus, Ga.; two at Tallassee, Ala.; one at Portman Shoals, S. C. The heights of these dams are not excessive, but their lengths are considerable. From an account given by an engineer who saw the lower dam at Tallassee after the flood had considerably subsided, it would seem that the dam would have stood the strain much better if it had been tied together with steel bonds. It might have cracked, but it probably would not have failed entirely.

Yours truly, H. F. Wilson, Jr.

315 Chalifoux Bldg., Birmingham, Ala., Jan. 4, 1902.

### Criticism of the Tallapoosa River Dam, Near Montgomery, Ala.

Sir: In the editorial note in Engineering News of Dec. 5 on the dam on the Tallapoosa River, near Montgomery, Ala., you say: "Doubtless many engineers will look askance at such a construction," referring to the concrete facing on the down-stream side of the dam.

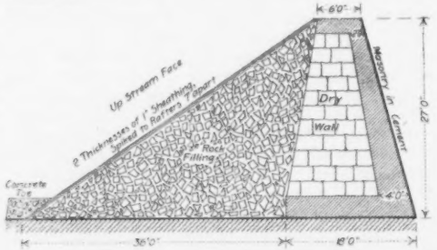
Will they not also "look askance" at the method of construction as described in the contributed article? I refer to the clause reading:

> In the central portion of the dam very large, rough stones were fitted together and all spaces filled with smaller stone, then packed with spawls so as to form, as nearly as possible, a solid mass of rock without mortar. After building up this material for 5 or 6 ft. grouting boxes were placed on the wall and the spaces between the spawls *were entirely filled with liquid cement, thereby insuring all horizontal and vertical joints being entirely filled and cementing the entire dam into one solid mass.*

(Italics are mine.) This is certainly a new departure in dam construction for a structure of such magnitude. Nearly all specifications for masonry structures, whether they are intended to hold back water or earth or to sustain a vertical load, prohibit most explicitly the above method of construction. It is doubtful whether much if any of the cement reaches the bottom of the vertical joints, and extremely doubtful whether any of it is carried into the bed joints. The writer has never seen it accomplished.

The thickness of the up-stream face, which was "laid carefully with cement mortar beds and joints," is not given, but assuming it is not more than 5 or 6 ft. thick, the water, under a head of 40 ft., will undoubtedly find its way through the facing wall, and then what will happen to the concrete facing? Evidently it is not heavy enough to withstand any such pressure and will be forced off.

That the weight of the structure has been increased 10% by this method of construction is also open to criticism. By what sort of mathematics was such a conclusion reached? I don't believe it. Possibly this structure will stand, like the Bear Valley dam, from pure cussedness.




An Example of Dam Construction in the South.

The writer recently came across the design of a dam being built in the South, a section of which is enclosed. When the engineer was asked what would become of the water if by chance it found its way through the sheathing on the up-stream face, he replied, "It would probably find its way out." Doubtless it will

Yours truly, John C. Temple.

200 West Walnut Lane, Germantown, Philadelphia, Dec. 30, 1901.

(In accordance with our custom we submitted proof of Mr. Temple's letter to the International Hydraulic Co., which is responsible for the design and construction of the dam. The company's reply follows.—Ed.)

Sir: We have read Mr. John C. Temple's criticism of the Tallapoosa River Dam. In all probability, Mr. Temple will find "many engineers will look askance at such a construction," referring to a water-tight wall having its central portion built up dry and then thoroughly grouted. Nevertheless, this is our method and the method of other engineers. "The proof of the pudding is in the eating." On Dec. 29, 1901, the day our contractors met with an accident, the unfinished closure and its coffer dam, which had a height of 5 ft. above the crest, were carried out. Before this took place the gage showed some 10 ft. of water going over the crest. The bulkhead was of the power house, built identically the same way, but without the concrete facing, showed no leakage whatever.

Your article referring to the liquid mortar being poured in on depths of 5 or 6 ft. is slightly in error; it should be at what might be called a course, though, strictly speaking, there was no course, the masonry being uncoursed rubble. The depth would not average 4 ft.

Mr. Temple states that

> nearly all specifications for masonry structures, whether they are intended to hold back water or earth or to sustain a vertical load, prohibit most explicitly the above method of construction. It is doubtful whether much, if any, cement reaches the bottom of the vertical joints and extremely doubtful whether any of it is carried into the bed joints. The writer has never seen it accomplished.

Perhaps "the writer" has never seen it properly tried but we have, and are well pleased with the results. Mr. Temple must keep in mind the fact that he is dealing with a rubble masonry wall, and that experience has taught engineers who have had something to do with water-tight walls, such as the one in question, that recourse must be had at all times to thorough grouting. The question then arises as to the method of doing it. Our reply to any of our confreres would be to try this method unless the wall is of ashlar.

Did Mr. Temple ever remove a large stone—in rubble work—once laid in the mortar bed? If so, he must be surprised to see the great number of voids present. Such voids in that method of laying stone cannot be reached by grouting.

Regarding what specifications call for, we wish to say that specifications change from time to time. Some years ago little or nothing was mentioned of concrete work apart from foundations. See what an important part it is now taking in large works. Little or no mention is made in most specifications respecting a vertical bond in the work; we must again make another departure, as we insist not only on a horizontal bond but a vertical one as well, with the view of making the structure monolithic.

Again, on the question of specifications: Many, on pile driving, compel a contractor to keep hammering at the piles till at the last blow with a 2,500-lb. hammer, falling 25 or 30 ft., a penetration of the pile into the subsoil of ¼-in. is allowed. We do not dispute the theory; it is right as far as it goes; but it does not go far enough. The inspector is often a young engineer, and his experience consists of what he sees in specifications—nothing more. How grand it would be for the engineering profession if more of its engineers did some contracting on their own account! What a helping hand they would find practical experience would give to theoretical knowledge!

Once, on some construction work, the writer was greatly bothered by an inspector—a confrere—insisting on the ¼-in. last blow, and in many instances damaging the piles. One fine day, at the noon hour, a pile was left in the leads about half driven, but a fall of some 25 ft. could still be had. After the pile had remained in this position for about an hour, the inspector was called upon to mark its height on the leads and the hammer was allowed to fall, with the result that no impression was made. The pile was marked O. K., but, greatly to the astonishment of the inspector, after several blows more, it went down at the rate of about 7 ins. to a blow. This simply goes to prove that the sustaining power of the pile depends upon the length of time it has been driven in the soil and its frictional resistance allowed to increase.

All this goes to show that additional work is required on some of our specifications. Regarding the question of cement, the trouble has been set aside, thanks to the efforts of the American and Canadian Societies of Civil Engineers.

Let us add, Mr. Editor, that it is practically impossible for an engineer to sit down in New York or Montreal, or even in Philadelphia, and conscientiously and correctly design a structure required a thousand miles off without carefully studying the local conditions.

We have cases of railway bridge piers being of brick, because no stone was at hand. The materials required for the manufacture of brick were there, so naturally brick was used.

The Tallapoosa Dam would not be suitable for Canada. There, in the event we were obliged to face with concrete, a greater thickness would be required to counteract the effect of ice.

Mr. Temple cannot understand how the weight is increased. Simply that the spaces between the larger stones are better filled with smaller ones and spalls. More stone and less mortar enters the wall, and by elementary mathematics "the conclusion is reached."

Quoting from Mr. Temple's letter, "Possibly this structure will stand, like the Bear Valley Dam, from pure cussedness." He avoids theoretical deductions regarding the stability of the dam against sliding at any joint or overturning. He has before him the section of the dam and an explanation of the method of construction, and he can readily get proof of its water-tightness by visiting the plant or writing to friends in the South. With all this data, he should be able to make his own calculations on a mathematical basis and not have to jump at conclusions. A careful computation by a reliable hydraulic

engineer will readily prove the safety and stability of the structure under discussion.

Your readers' attention is called to a sketch which Mr. Temple encloses of a dam being built in the South. We certainly would not care about adopting the section as a standard one. We are inclined to think, however, that Mr. Temple is again confused, and that in this case he has the "cart before the horse," for undoubtedly better results would be obtained if this section were reversed with respect to the flow of the stream.' The sheathing in question certainly cannot be made water-tight, but if on the down-stream side will act very nicely as an apron.

Healthy criticisms are at all times desirable, and should be encouraged; but there is no necessity for resorting to hurried conclusions without knowledge of the facts, nor expressing the opinion (absurd though it be) that if a structure does stand it will be through "pure cussedness."

Yours truly,
International Hydraulic Co., Engineers.

40 Wall St., N. Y., Jan. 10, 1902.

'We are assured by Mr. Temple that the section accompanying his letter agrees with the account of the structure which was furnished to him.—Ed.)

## Place the Isthmian Canal Work in Charge of an Engineering Commission.

Sir: As your paper is the leading exponent of the engineering profession in the United States, engineers look to it to watch over the interests of the profession at large and use all legitimate means for its advancement.

You are aware that the bill for the Nicaragua Canal has passed the House and will soon be considered by the Senate; the House bill practically provides for doing the work under direction of the Secretary of War, which means that the engineering work will be under the control of the army officers of the United States Engineer Corps.

It would seem advisable that the engineering profession outside of the United States Army should be considered in connection with the work of designing, supervising and constructing this enterprise, which will probably be the most important engineering work carried on during the Twentieth Century. It will call for the highest engineering, executive and administrative talent; but there is no question but that the engineering profession can furnish the requisite talent, rare though it may be.

It is suggested that you take this matter up editorially and champion the cause of the profession. There is perhaps no business interest in the United States that would idly let an opportunity of this kind go by without proper representation. It is a question of so much importance to engineers that it would seem that some concentrated effort should be made, either through the American Society of Civil Engineers, or the local engineering societies, to secure proper recognition of the profession. It also seems that the members of the profession having a personal acquaintance or any influence with the Senators and Representatives should use that influence both individually and collectively to properly present the importance of this subject to our legislators at Washington who now have the matter under consideration.

A movement of this character can be carried on without any reflection upon the eminent engineers connected with the United States Army. It is, however, un-American to place this important work under the charge of a single small class of the profession and to bar out from all large responsibilities all other members of the profession, among whom are men who have spent a lifetime in fitting themselves for such work. It was perhaps at one time proper to employ army engineers for Government works, in times of peace and during the early years of the Republic, when the profession of engineering was in its infancy in the United States and educated engineers were rare. The situation to-day, however, is entirely different, and it would now seem more fitting that the United States Engineer Corps should confine its duties to those of a military nature; or at least that the graduates of Troy, the Massachusetts Institute of Technology, Cornell and numerous engineering schools of high quality should have at least the same chance for responsible positions on Government works that is given to the graduates of West Point, where an engineering education is a mere incident in the military training.

At present the line of demarcation in Government works between the army engineer who has been educated at West Point and the civil engineer who has been educated at the best engineering schools in the country, is as well defined as the line of demarcation between the commissioned and the non-commissioned officer in the ranks of the regular army, which we all know is a gulf almost impassable, particularly in a social way. However, it is possible for the non-commissioned officer at times to cross this abyss, but to the civil engineer assistant to the United States army officer, it is absolutely impossible.

The engineering profession of America should at least demand that its members should have an equal show for the positions of trust and responsibility in connection with the design and construction of the Nicaragua Canal with the officers of the United States Engineering Corps, and an equal opportunity for promotion and advancement in connection therewith. The first step to put this suggestion into effect should, of course, be the placing of the supervision of the design and construction of the canal under a commission composed of eminent civil engineers.

Very truly yours, John F. Wallace.
Asst. Gen. Manager, Ill. Central R. R.

Chicago, Ill., Jan. 10, 1902.

(This journal has repeatedly and strenuously urged that the construction of the Isthmian canal should be placed by Congress in the hands of a Commission, to be appointed by the President. Such a Commission should preferably be composed either entirely or largely of engineers of high standing, such as those selected to form the present Isthmian Canal Commission.

We have urged this upon the attention of Congress, not so much in the interest of the engineering profession as in the interest of the country. The United States needs the services of the ablest engineering talent that it can secure to direct the construction of an Isthmian canal vastly more than the engineers need the opportunity to direct the work.

We have not the slightest intention of reflecting on the talent and ability of members of the Corps of Engineers; but those among them fitted by years, ability and experience to fill the highest and most responsible positions on engineering work are comparatively few in number, and have at present as much as they can do in the conduct of River and Harbor and fortification work now in progress. Neither these men nor the United States can gain anything by saddling them with the additional great responsibility of an Isthmian canal. In fact, many of these officers, if assigned to duty there, would find it hard to get along with their inadequate and unjustly small pay and the necessary expense of living in the tropics. As for the employment of civilian engineer assistants, under present law and custom, the pay of such men is limited to a maximum of about $3,500 a year—an amount manifestly inadequate to secure the class of engineering talent which the work on the Isthmus demands.

Mr. Hepburn refused to provide for a Commission in his Nicaragua Canal Bill, which has just passed the House, on the ground that he was averse to manufacturing berths for superannuated politicians—as if it were to be expected that President Roosevelt would select men of this sort in making appointments to such a Commission! The Hepburn bill, however, provides that the President "may employ such persons as he chooses at such compensation as he sees fit," which might be construed to give him authority to create a Commission to take charge of the work.

Congress at present is concerning itself far more over the battle between the rival routes than over questions of procedure in construction; but Mr. Wallace's suggestion that engineering societies should memorialize Congressmen, and especially Senators, upon the creation of an engineering Commission to supervise the work appears to us an excellent one.

Let us urge, however, that the public welfare and not the honors and emoluments of the profession itself be made the ground of every such appeal. The lawmakers at Washington are well calloused to the petitions of those who seek something for themselves; and it will be much more dignified as well as more effective if organized bodies of engineers urge the interest of the public in securing the best expert talent for the work, to the end that it may be done in the best and most economical manner.—Ed.)

## The Physical Condition and Safety Under Present Loads of the New York and Brooklyn Bridge.

Sir: In your issue of Oct. 10, 1901, is published the complete report on "The Physical Condition and Safety Under Present Loads of the New York and Brooklyn Bridge" by Messrs. Duryea and Mayer. The instructions to the examining engineers were chiefly to ascertain "whether or not the capacity of the bridge is overtaxed at the present time." The following comments therefore refer to that part of the report relating to the above instructions, as this is mainly of interest to your readers. Nothing is said in these instructions as to a criticism of the original design, whether a better structure could be built at the present time, either as to design, material or workmanship; whether improvements could be made in details, or whether the capacity of the bridge could be increased. It is plain that the report goes beyond the instructions in all of these matters, as it declares the bridge safe at the present time.

In order to determine whether the bridge is now overtaxed, it becomes necessary to subject it to a complete and thorough inspection as to its present condition and to ascertain the qualities or properties of the materials of which it is composed, and also the stress to which it is now subjected in its principal supporting members. The report states that the inspecting was done in an incomplete manner, and it shows that nothing was done to ascertain the qualities of the material.

After a "hasty examination," it is declared "that the saddles are rigidly fixed and cannot move," because they did not move during the short period of observation. These saddles cannot move until all resistances are overcome, which happens only under great differences between the loads on the land and the main spans, which rarely occur in August, the time the examination was made. The presence of "dust, paint and rust near the outer saddle rollers" is accepted as proof that the roller beds are blocked by similar material. No attempt was made to determine the condition of the intermediate rollers or of their bearing surfaces because "they were not accessible." It is stated that as some stays bear on the tops of the towers this prevents all motion of the saddles. This is mere assumption, for it is a fact well known to those who have had an opportunity of examining similar suspension bridges that this condition of the stays does not at all prevent motion of the saddles, but that the stays actually slip on the masonry and on the saddles. This was demonstrated by the Cincinnati and Covington Bridge during its reconstruction. With these facts before us, and which were obtainable, all of the published calculations and conclusions based on the fixedness of the saddles are hypothetical and should be rejected as of no value or reliability. The report gives no proof that any attempt was made to determine accurately the relative positions of all of the saddles or of the cables.

No reference is made in the report to several sets of existing top-chord joints in the stiffening trusses, the slip joints at the center of the main span only being mentioned, and the calculations of loads on suspenders and cables are based on the assumption that all chords of the trusses are continuous throughout from slip joint to towers, and as having a true hinge at center. As this assumption is not in accord with existing conditions, all calculations based thereon are also unreliable and incorrect.

Nowhere in the report is there any evidence showing that any attempt was made to determine the properties of the material of the bridge, and the working stresses adopted by the reviewers prove that they assumed the trusses and floor beams to consist of what is at the present time called structural or low steel, which has a yield point of 35,000 lbs. and a tenacity of 60,000 lbs. per sq. in. and which is commonly subjected to working stresses of 14,000 lbs. per sq. in. by the most cautious engineers. It is also stated that the "floor beam chord stresses are 15,500 and the working stresses should be only 14,000 lbs. per sq. in." While these chord stresses are correctly calculated, it is a well known fact that the working stresses commonly allowed in steel having a minimum tensile strength of 70,000 lbs. per sq. in. are as high as 16,000 lbs. per sq. in.

Many tests of full-size members in the precise conditions in which they were used in floor beams with the rivets in place made by me at the bridge shops in 1880 showed the steel to have a yield point of over 55,000 lbs. and a tenacity of over 80,000 lbs. per sq. in. The specifications prescribed tests of large sample bars, and while these also showed more than the prescribed minimum, the steel as rolled into the shapes used in the structure showed the higher results given above, even after allowing for the friction of the testing machine. Now, if it is considered safe to use a working stress of 16,000 lbs. per sq. in. when the material has a yield point of 40,000 lbs., it is certainly equally safe to allow a working stress of 25,000 lbs. when the material actually shows a yield point of 55,000 lbs. and a tenacity of 80,000 lbs. per sq. in. in full-size members. The load factor under such stresses would, then, be the same for both kinds of material, while the factor of safety, based on tenacity, would in the first case be 3.75 as against 3.2 in the latter. If it be now considered that 16,000 lbs. is frequently the working stress in ordinary railway bridges, where the failure of a single member means the collapse of the structure, and where accidents due to derailment or breakage of axles or wheels must be taken into account as possible causes of injury and collapse, and provision made for them, it will be seen that these considerations do not hold good in a suspension bridge, especially when of the gigantic proportions of the one under review. Injury to trusses or floor beams in this case does not in any way affect the safety of the structure. It is furthermore common practice to reduce factors of safety in structures of large dimensions, in which it is not likely that the assumed loads used as bases of calculations are ever reached, much less exceeded. Only those members possibly subject to unusual or excessive stress and impact are in such cases calculated on the basis of a larger factor of safety.

The reviewing engineers have had no experience in the construction, maintenance or repair of suspension

bridges, and their assumptions, comparisons and opinions are based almost entirely on their experience with truss or cantilever bridges and masonry. The report asserts that "the science of bridge building was then (in 1876-1879) very much cruder than now, and very little attention was paid to secondary stresses." This statement no doubt applies to truss bridge building, but anyone familiar with the whole literature on the subject of suspension bridges of the type under review knows that it is a mere assumption and does not apply to them. The statement that "many of the features which now seem self-evident errors have been realized as such only from the lessons taught by their use in early bridges" does not apply to suspension bridges at all, but to truss bridges alone. Of the former there are but two, viz.: the Cincinnati and Covington Bridge and the New York and Brooklyn Bridge. Neither of these has been modified in its details, and there are and were no early bridges of this type and size from which lessons could be and were learned. The reconstruction of the Cincinnati and Covington Bridge was not necessitated by "self-evident errors" found in it, but because it became necessary to more than double its capacity and to carry heavier loads. Its essential details are now practically the same as formerly. Only a few engineers, and these not the reviewers, have had any experience with these two bridges, and to them the bridges have demonstrated that they contain but few errors and have given great satisfaction. Numberless truss bridges have been worn out, have failed in service and have been replaced by better designed structures in the same period of service.

It is also stated that "our investigation shows that the stresses in cables are far in excess of those considered safe by engineers." This again is a statement based entirely on assumptions, which I shall disprove by reference to observations made at the time the bridge was building. While the specifications called for wire having a minimum tenacity of 160,000 lbs., actual tests of the wire used in the bridge made by myself showed an average yield point of about seven-eighths of the tenacity, which was 180,000 lbs. Now, in determining the safety of this bridge at the present time, the latter figures, known to be correct, should be used, and not those assumed in the report. Admitting, for argument's sake, the total stresses in some of the cable wires to be 71,000 lbs., as calculated and stated, and bearing in mind that these cables cannot be subjected to impact of any kind or to unlooked for extraordinary stresses, this stress by no means produces "a very serious condition—that of a stress in the cables much above the safe limit, but with its exact amount unknown." Moreover, this calculated stress is based on a fictitious assumption, which does not apply to this bridge or to any bridge of this type. The assumption that the cables bend locally at the center, due to deflection of the trusses at high temperature, being drawn down by them, is not a condition existing in this bridge, and is purely arbitrary. Because of the actual design in which no "hinge" is provided at the centers of either the main or the land spans, the secondary stresses assumed to be produced in the cables cannot occur under any possible loading. The slip joints do not transmit longitudinal forces, but resist vertical forces alone. Why does the report invariably assume imaginary stresses of the greatest amount and working stresses which are always lower than those permissible in the materials actually in use in this bridge? As this was the first all-steel bridge, every possible effort should have been made to learn everything about the materials used in it before presuming to say whether any one of its members is now overloaded.

It is also stated that where joints are not supported by stringers "the rail itself must be the main carrier of the load, which is a condition outside the limits of good practice;" and again, "this is extremely bad practice." The reviewers forget when making this severe criticism that in all surface roads the rail is the sole carrier of all loads and that the size of rails are proportioned thereto. It is entirely proper to make such a perfectly safe and well-made steel shape as a rail do its full share of the work in this structure as is done in many other cases.

The report is based on other assumptions relating to the design. It assumes that the suspender connecting bolts could and should have been made with upset ends, not knowing that when these rods were actually manufactured, in 1880, there were no upsetting machines in existance capable of doing such work on the material to be used. Rods could have been upset a few inches, but not the length required in this bridge. It is well to state here that the first machines for upsetting large steel bars and for making eye-bars were designed and constructed for this bridge, none being in existence theretofore.

The most peculiar omission in the report is the absolute failure to determine the cause of rupture of the nine center suspender rods under the north cable, and to merely assume that wind pressures produced their failure, when it is repeatedly stated that the wind stresses were not calculated. All statements about transmission of wind stresses are based on conditions existing only in truss bridges. The writers of the report apparently fail to see that the cables do not remain fixed in their positions, but that they move laterally with the wind, and that the floor and under-floor wind stays provide solely and amply for wind stresses. The lateral displacement of cables does produce very slight additional stresses in them, which are, however, negligible. If the cables did not swing laterally under wind pressures, these would produce stresses in the short suspenders, and the latter would transmit them to the cables. This is, however, not the case in this design. The report repeatedly states that seven suspenders broke on the day of the accident and that two had been broken for a long period before that time. One of its authors was on the bridge on the day after the accident while the broken rods were being removed and failed to see what I pointed out, viz.: that only two suspenders showed new fractures, while all the others were covered with rust and dirt, one as much as ⅛-in. thick, giving evidence of ruptures of long standing. One of the rods last broken showed a new crystalline fracture across its entire section, while the only other showing a new or recent fracture also showed successive previous ruptures and a sector of about one-eighth of the total area with bright crystalline surface. These two newly broken rods were both located east of the center of main span. Now, if wind stresses caused failure of these suspenders, why did none of those under the other cables fail or even show signs of failure? They were certainly exposed to nearly similar conditions.

The real cause of the failure of the suspenders must be sought in another condition, and one which is mentioned in the report, but not defined as a cause in it. The report states:

It is a fact, however, that the distance from cable to floor beam was and is greater at the northernmost cable than at any other, this cable being now about 7 ins. higher than the others. The reason for this is apparent from the fact that for some years past the span of the northernmost cable has been longer by several inches than those of the others, thus causing the center of this cable to hang at a higher elevation.

Immediately preceding this information is the following statement:

Some of the newspapers reported that when the broken rods were discovered measurements showed that the floor beams at these rods had sagged several inches at their upstream ends, or that the cable had lifted that amount. This is manifestly impossible, as a small part of such a rise would bring the cable above the line joining the two first unbroken suspenders, and such a sag would cause the ruin of the floor beam.

The fact is, as noted by myself on the day after the accident, that the north truss to which the floor beams are attached had sagged several inches, forming quite a depression at the slip joint, in spite of the above positive statement that this is "manifestly impossible." Although the report clearly shows that the north cable hangs 7 ins. higher than the others, and that the suspenders have not been lengthened correspondingly, no attempt is made either to calculate the increased loads which this condition would impose on the middle suspenders, or to figure the stresses in the floor beams. The assertion that the lifting of the cable several inches is "manifestly impossible," because it "would cause the ruin of the floor beams" is again based on the assumption that this suspension bridge is as rigid and inflexible as a truss bridge, and disregards the absolute flexibility and compensating capabilities of the former. Of course, if the floor beams were to bend "several inches" in the distance between two cables (about 37 ft.), they might break; but a sag of "several inches" at one end of the floor beams by no means indicates distortion of them by that amount; it simply means a displacement of the ends of the floor beams relative to the position of the cable when disconnected from it, due to flexure of floor beams and to readjustment of the relative positions of the three other cables and of the north trusses. I shall not now state why the north cable was and is hanging higher than the others, but its peculiar position did produce unusual loads and stresses upon the suspenders near center of the span. This was the primary cause acting during a long period of time (probably three years), which caused the failure of all the nine solid suspenders, including the last two which failed suddenly on July 24, 1901. The fractures of the others showed successive partial failures. This is a defect in the bridge existing at the present time, without, however, affecting its safety, which can be remedied; but the effect of which has not been determined in the report. It is still unexplained why some broken suspenders were not found under the south cable at center of span, where I should expect to find some.

Trusting that I have added some facts relating to the history and the present condition of the New York and Brooklyn Bridge of interest to engineers, I am,

Respectfully, Gus. C. Henning.

220 Broadway, New York city, Dec. 31, 1901.

## Mr. Hildenbrand's Reply to the Report of Messrs. Duryea and Mayer on the New York and Brooklyn Bridge.

Sir: Of technical literature, recently published, there is probably none of wider interest or which has created more commotion among engineers and the general public, than the report of Messrs. Duryea and Mayer on the condition of the Brooklyn Bridge, which was published in Engineering News, Oct. 10, 1901.

The examination of the bridge by the two engineers named and their subsequent report was instigated by the District Attorney of New York city, with a view of ascertaining whether the strength of the bridge was overtaxed under the present traffic, whether any deterioration had taken place, and whether any facts existed which would show criminal negligence on the part of the persons in charge of the bridge.

The questions of the District Attorney and the importance of the subject itself, involving, as it does, the safety or the peace of mind of millions of people, furnished ample material to an engineer for sincere investigation and for an exhaustive and decisive report on the result of his labors. The publication of such a report was heralded in advance by the daily press, raising the expectations of everybody interested in the Brooklyn Bridge to a point almost of impatience. It was, therefore, the more disappointing when finally these expectations were not realized. Engineers failed to find in the report a fair criticism of the structure, based on exact observations and calculations, and the public was more mystified than ever as to the condition of the bridge, failing to find it pronounced either perfectly safe or absolutely unsafe. I agree, in this respect, with the public and believe that the report lacks the proper decision.

If the bridge is safe for the traffic it accommodates at present, it should be said so very decisively, and the minds of many timid travelers should be relieved from anxiety; if, on the other hand, the bridge is absolutely unsafe and dangerous for public use, it should be told unreservedly, if the examiners have conscientiously found it so after a critical consideration of all really existing, and not arbitrarily assumed, circumstances!

From what the examiners actually observed and reported, with strict adherence to the instructions of the District Attorney, they should, properly, have brought in a clear verdict of safety for the bridge; but they volunteered to do more than was demanded by the District Attorney and they succeeded in discovering one, in their view, great defect in the original design of the bridge, in the form of an unknown and unprovided for "secondary stress" in the cables. With what justice and correctness this assertion is sustained, I shall discuss later on, but it seemed sufficient to the examining engineers for raising the danger cry and for throwing, either with or without special intention, a whole community into anxious doubts as to the safety of a structure which for the past 18 years has been trustingly made use of by millions of people, in blissful ignorance of this dangerous defect. Of course, the same conditions existed 18 years ago as to-day, and if the examiners' report had been published at that time and had found as many believers as it, seemingly, does now, the citizens of Brooklyn and New York might have been deprived of the comfort which the bridge has afforded to them for the age of half a generation!

The "secondary stresses" give also reason to the examiners for severely arraigning the designer and the builders of the bridge, plainly charging the former with ignorance and incapacity as to properly planning and calculating a suspension bridge. They do not say this in precisely these words, but if it is not the meaning of several paragraphs of the report, these paragraphs have no meaning at all. The examiners themselves acknowledge such a meaning, else they would not have taken occasion to excuse their imputations with the declaration that they were only criticising the design and not the designer. This, of course, is mere sophistry, because a design does not make itself, but needs a designer who alone is responsible for the design or plan.

However, an excuse is not necessary, because the builder of a great public work expects, and even solicits, a fair criticism from experts, whether favorable or unfavorable, and suggestions for improvements are not only legitimate, but are, or should be, thankfully received. The report, submitted to the District Attorney, is not of that kind. The examiners chose to give to their criticism an exclusive tone of fault-finding, which cannot be misunderstood. The reader also cannot help noticing that the writers of the report assume an air of superiority of knowledge and judgment, from which no appeal appears to be possible.

They do not recognize that things can be looked at from two sides, and that, in matters of opinion, their own views are not necessarily entitled to more consideration than those of other engineers.

While these remarks about the general tone of the examiners' report have no direct bearing on the conditions of the Brooklyn Bridge, they are intended to have an indirect bearing on the judgment of the public in regard to the relative merits of different treatises on the same subject. In a complex structure, like the Brooklyn Bridge, where the conditions of loading and the effect of the loads on the members of the bridge are not absolutely or rigidly fixed, it is natural that more than one assumption of condition is probable, and more than one method of calculation possible. Even, if the results of the different calculations should agree tolerably well, the opinions, as to the amount of admissible stresses or as to whether the danger point has been reached or not, may differ. In each case the judgment of the public should principally be guided by the merits of the argument, or, if any arbitrary preference should become necessary, it ought to be rather on the side of those whose past record can prove the greater experience in the construction of suspension bridges, than on the side of those with lesser experience!

In the following discussion I have attempted to review

the public criticism on the Brooklyn Bridge, with the sole object in view of restoring to the community that confidence in the Brooklyn Bridge which is properly due to it, and which the two examining engineers have, according to my opinion, unnecessarily and wrongly disturbed. My arguments will be based partially on a strictly scientific basis, and partially on precedents or common-sense reasoning, in order to prove, on one side, to engineers by calculations and figures that the examiners' conclusions are inexact or exaggerated; and, on the other side, to convince the non-professional public that the fears about the general safety of the Brooklyn Bridge are entirely unfounded.

SOME EARLY CRITICISMS OF SUSPENSION BRIDGES.

Before proceeding with this discourse, I beg to relate a few significant facts of past history.

In 1876, or 22 years after the Niagara railway suspension bridge had been in successful operation, Mr. Edward Wasell, a bridge engineer and inventor of an improved suspension truss girder, published a pamphlet criticising the construction of the Niagara bridge. He proved by exact mathematical formulas that, under the assumption of a certain combination of circumstances, the safety factor for the cables was barely as high as 2.2, inasmuch as under the extremes of temperature, one pair of cables had to carry not only the entire live and dead loads of the bridge, but, in addition, also the dead weight of the other pair of cables. The managers of the Great Western Railway Co. of Canada, alarmed by Mr. Wasell's gloomy prediction of seeing, at any moment, a passenger train engulfed in the frightful abyss of the whirlpool, requested Col. W. A. Roebling, whose father was the builder of the bridge, to examine Mr. Wasell's criticism, and to advise the company what to do. In a pamphlet, entitled "A Reply to the Recent Criticism made by Mr. Edward Wasell on the Niagara Railway Suspension Bridge," Mr. Roebling demonstrated that the kernel of truth contained in Wasell's reasoning and calculations would dwindle to insignificant proportions if the problem were considered in its correct light. He proved by exact figures that, under the actual loads, the factor of safety was "4," as originally calculated, and that the total excess of stress by a change from mean to extreme temperature amounted to $35^{16}/_{100}$ tons per cable, or only 1½% of the strength of the cable. This result was obtained by taking account of the elastic properties of the wire which Mr. Wasell had neglected in his computations. As a fact, there was never an accident to any train on the Niagara bridge. Twenty years after Wasell's publication, and after having done service for 42 years, the bridge was finally replaced by another structure, not on account of any weakness in the cables, through either primary or secondary stresses, but on account of its small capacity to comply with the increased demands of modern travel. Mr. Wasell's pamphlet has a striking resemblance to some parts of the examiners' report, as will be pointed out later on.

In 1894, the New York & New Jersey Bridge Co. submitted to the Secretary of War for approval a plan for a railroad bridge across the North River, consisting of a cantilever of about 2,600-ft. span, resting on a pier in the middle of the river. Upon the protest of the Chamber of Commerce against the obstruction of New York harbor by piers, the President of the United States appointed five highly reputed bridge engineers and mathematicians to decide whether a suspension bridge with a single span of 3,300 ft. would be feasible for railroad purposes, or not. One of the examiners, who is known as the designer of the above-mentioned cantilever plan, testified against the idea of a single-span suspension bridge as impracticable and not economical. This testimony was given notwithstanding the fact that a practical and well studied design for such a bridge, originated by Mr. Gustav Lindenthal, was already in existence, and had been described in various scientific papers, and that a company was incorporated and working to raise funds for carrying out that project. These facts were given by Mr. Lindenthal before the committee in opposition to the cantilever scheme. The Chamber of Commerce retained also one of the engineers, who had assisted in the construction of the Brooklyn Bridge, as its engineering counsel for arguing against the construction of piers in the river. He was able to demonstrate to the committee that, with certain new features proposed, a railroad suspension bridge of 3,300-ft. span, complying with all modern demands, is practical and also economical. The general plans and calculations for such a bridge submitted by the engineer of the Chamber of Commerce, were, after careful consideration and much critical discussion, substantially adopted by the government board of engineers and recommended by them as the best type of bridge for spanning the North River. The gentlemen who constituted the committee appointed by the President of the United States, were: Maj. C. W. Raymond, U.S.A.: Messrs. Geo. S. Morison, Theodore Cooper, W. H. Burr, and G. Bouscaren, Ms. Am. Soc. C. E.; all engineers and bridge builders of great experience and of the highest reputation.

The report of this board of engineers was approved by the Secretary of War, which settled the pier question of the North River bridge. The designer of the cantilever went to prepare new designs for spanning the North River with a bridge of the suspension type, to which reference has repeatedly been made in the report of the examiners.

The writer of this discourse, who had the honor of being the engineering counsel for the Chamber of Commerce, went to build, or rather rebuild, another suspension bridge over the Ohio River, at Cincinnati, which, next to the Brooklyn Bridge, is the largest in existence. The Cincinnati bridge has a span of 1,056 ft., and is constructed precisely on the same general principles as the Brooklyn Bridge, but is considered to have a number of improvements, which fully obviate all difficulties, so far encountered, with some parts of the Brooklyn Bridge. This is not the place to describe the Cincinnati bridge, but it demonstrates that the builders of the Brooklyn Bridge agree with the examiners that their work is not perfect, and that improvements are possible and perhaps advisable; but it is far from admitting that a few minor defects make the whole bridge unsafe for use! It may be asked: What have past events, or the Niagara, North River and Cincinnati bridges to do with the Brooklyn Bridge? It is true, they are not directly concerned in this discourse, nor do they remove secondary stresses from the cables, but I believe that the facts, as told, teach the following lessons: (1) That engineers are sometimes mistaken when they draw hasty conclusions from mere abstract mathematical calculations. (2) That the assertions of the examiners, charging the engineers of the Brooklyn Bridge with having neglected to improve their knowledge, are unfounded inasmuch as the same engineers have given practical evidence of their ability to progress in the art of suspension-bridge construction, and also have made new discoveries in the properties of steel wire, which are of more practical value than the theoretical derivation of formulas for determining uncertain stresses which may never happen, or, at best, may happen not more than once in a lifetime. It must be supposed that the examiners are not acquainted with the true properties of steel wire, else they would have drawn different, and much more favorable, conclusions from their results of the secondary stresses, even if the latter existed to the full extent as given.

PRESENT CONDITION OF BRIDGE.

Returning now to the report of the examiners, I shall confine my review and criticism principally to those points in the report which deal with the original plan, and which throw doubts on the safety of the bridge in consequence of supposed defects in the design. About the present physical condition of the Brooklyn Bridge I shall make no comments. I have not examined the structure; but, if the inspection of the examiners can be trusted, the bridge must in every respect be declared to be as safe as it ever was. The inspectors found that the steel work of the bridge is "quite free from harmful rust;" that "no important deterioration exists;" that "the structure was kept unusually well painted;" that "all rivets tested were found tight and in good condition;" that "the strands of the cable within the anchorages, and wherever else examined were dry, well painted and in good condition;" that "the wearing of some parts of the superstructure had as yet no serious effect on the strength of the structure;" and that "the masonry of the towers, though examined with a powerful marine glass, shows no visible signs of deterioration," and that "the appearance of the masonry indicates that it is of excellent quality."

These different statements are occasionally contradicted, and the engineers in charge of the bridge are severely arraigned for neglect of duty, but in view of the facts, as quoted above verbatim from the report, it is evident that the negligence of the guardians of the bridge could not have been great, and that the latter do not deserve any serious rebuke.

The only place where the condition of the bridge has materially changed from its original condition is on top of the towers in the cable saddles, which were constructed to move on rollers, but which the examiners found to be immovable, on account of the accumulation of dirt and rust, and in consequence of some of the rollers having assumed an oblique position. It is certain that, with proper care and supervision, the saddles could have been kept free from dirt and rust, but it is improbable that the best of care could have prevented the rollers from taking an oblique turn. The fault, if any, for causing this condition, must be in the original design. However, the examiners do not lay much stress on this circumstance, because they decided that the fixedness of the saddles, which at first appeared to be a serious defect, is in reality a benefit to the bridge.

ASSUMED CAUSE OF THE ACCIDENT.

There was also a defect, somewhere, for causing the rupture of several suspender rods last summer. Either extraordinary and unforeseen forces must have been at work, or there was a lack of proper care, or there was a mistake in the design which made the occurrence unavoidable. Different engineers have given their opinions as to the probable causes of that accident; the writer has expressed his views in Engineering News of Aug. 8, 1901. There is no reason for changing these views, but I would add that it is probable that the crystalization of the steel through frequent and violent impacts, when bridge trains run at the rate of 40 miles per hour, may have made the rods brittle, and liable to break easily under bending strains to which they were undoubtedly subjected. There is reason for this supposition in the fact that all broken rods were short and near the center of the span where the worn condition of the slip joint gave occasion for violent vibrations. Speaking about this accident and its causes, the examiners rebuke the engineers in charge for not keeping the trunnions oiled or for neglecting a rigid and frequent inspection; but they finally exonerate the latter from all blame by declaring decidedly that the rupture was caused by lateral strains from wind pressure and that "the excessive stress on the middle suspender rod are inherent in the design," and that "an entire change in the design is necessary to transfer this wind pressure safely." As the wind pressure and wind stresses will come up again under the head of secondary stresses, I will comment on this subject here.

WIND STRESSES IN CABLES AND SUSPENDERS.

The examiners are very positive in their statement of blaming the design for causing excessive stresses in the suspenders and cables. I am just as positive in declaring that the examiners are mistaken. As in all such matters, there is a small kernel of truth in the examiners' assertion, because the wind pressure unquestionably does increase the tension in cables as well as in the suspenders, but this tension is by no means "excessive," nor dangerous, and, according to my opinion, it is certainly not the cause of the rupture of the rods. The examiners fail to give a satisfactory explanation of how an excessive bending stress in the suspenders becomes possible; they also fail to give any figures as to the amount of these stresses, and do not even indicate a method for calculating them. The examiners simply state: "we know that the wind pressure to be transferred to cables at center of bridge is large, though we have not calculated the actual amount." A sort of explanation is attempted in the following sentence: "The amount of wind pressure transferred by these suspenders is dependent on the horizontal curve of the stiffening trusses." If the examiners had said that the amount of wind pressure transferred is dependent on the *difference between* the horizontal curve of the stiffening truss and that of the cable, they would have come a little nearer to the truth; because it is evident that if the side motion of cable and truss is the same, there is absolutely no wind pressure to transfer. The only increase of tension in the cables and suspenders in this case is due to the difference between the dead load and the resultant force of dead load and wind pressure. This difference is too small to deserve any practical consideration. As far as I understand the examiners' idea of the wind pressure and its effect on suspenders and cables, they consider the cables as fixed and the floor with its trusses movable, about as illustrated in Fig. 1. Let A represent the floor of the bridge; C the cable; and, B a beam fastened to the floor and leaning against the cable. Now, let the wind press against the floor and the floor will assume the position $A^1$ and the upright beam the position $B^1$, as indicated in the dotted lines. If this assumption were true, there would indeed be a large bending stress in beam B, which represents the suspender, and a large extra pressure on the cable. This is a very erroneous view, and shows that the examiners have never observed the action of a suspension bridge under the influence of a heavy gale. I have been on the Brooklyn Bridge when the force of the wind made it impossible to stand without holding on with a firm grip to some fixed object, and, to my eye, the stiffening truss on the windward side of the bridge appeared to be perfectly straight, while the cable made a visible horizontal curve inward; in other words, what took place was just the opposite to what the examiners assume to be the case.

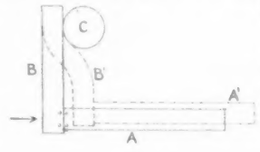


Fig. 1.

The effect of the wind pressure on the superstructure of the main span was very carefully examined and calculated at the time the bridge was constructed. In addition to the inherent stiffness of the floor and to a complete system of wind diagonals, similar to the lateral bracing in any truss bridge, there are two parabolic horizontal storm cables and an extensive system of underfloor horizontal stays for counteracting the horizontal deflection of the floor, caused by the wind. In other words, the bridge is, horizontally considered, the same kind of a suspension bridge for resisting the wind forces, as it is a vertical suspension bridge for sustaining the loads.

A wind pressure of 30 lbs. per sq. ft., under the supposition that it presses on all six trusses with equal force, will cause a side deflection of the floor of 1.2 ft. at center of span. This result was obtained by assuming a maximum stress for parabolic cables and underfloor stays of nearly one-half of their ultimate strength. While this stress might appear large, if applied to a truss bridge where the failure of a single rod would cause the collapse of the whole bridge, it is a perfectly safe assumption for a suspension bridge, where the destruction of the whole lateral bracing would merely cause large and vehement lateral oscillations, without endangering the general safety of the bridge. This is a point to which I have repeatedly called attention, and one which the examiners always neglect to recognize, namely, that a suspension bridge is in permanent equilibrium, while a truss bridge has only an unstable equilibrium, which means, that its equilib-

rium is not restored when once disturbed, causing the collapse of the truss, while the equilibrium of a suspension bridge restores itself, no matter how large the disturbance may have been. The side deflection of the cables under the same wind pressure, with due consideration of the "cradling" of the cables, amounts to 1.5 ft. This result was corroborated by actual observation, as mentioned above, and it shows that there is no wind pressure to be transferred to the cables by the suspenders, while the pressure transferred from the cables to the floor is too small to be significant.

The examiners' imputation, that an inherent defect in the design is the cause of excessive bending stresses in the middle suspender rods, is not sustained by facts or experience. The same design has been used for the past 50 years in all large and small suspension bridges, and there never has been a case similar to the recent accident on the Brooklyn Bridge. We can infer from this fact that the design, as demonstrated by its utility for many years, is good, because there is no wind pressure to be transferred to the cables by transverse action of the suspender rods. Whatever pressure is transferred is by a direct pull in the direction of the suspender.

## SUMMARY OF EXAMINERS' REPORT.

Outside of the chapter referring to the broken suspender rods, the substance of the examiners' report and the result of their investigation, can be condensed to the following statements:

1. The bridge is well preserved; not deteriorated by rust or wear and tear, and, therefore, as strong as ever.
2. The bridge is not overloaded, the combined dead and live load being practically the same as assumed in the original design. The wheel loads or concentrated loads are also the same or very nearly the same as originally assumed for the calculation of the details; hence,
3. The bridge is as safe as when first completed.
4. The bridge was never safe on account of certain secondary stresses in the cables, unknown to the builders of the bridge, and on account of some defects in the design and of excessive stresses in the floor systems!

These four statements are almost literally the words of the verdict of the examiners with exception of the third statement which is my own.

The examiners did not state that the bridge is as safe as ever, but they *should have* said so, as a simple consequence of their first and second statements.

The report contains also some purely theoretical investigations; one referring to the deformation of unstiffened cables under loads or changes of temperature, and another to the maximum stresses in stiffening trusses, all of which may be of interest to the student of suspension bridges, but has no bearing on the investigations of the examiners, and has absolutely nothing to do with the safety of the Brooklyn Bridge.

I shall, therefore, not comment on this part of the report, but consider only the examiners' statements in regard to supposed defects in the design and especially in regard to the "secondary stresses" in the cables. These "secondary stresses" seem to be the keynote of the whole report and more importance is ascribed to them than to anything else in the whole investigation.

## SECONDARY STRESSES IN CABLES.

Let us see with what right this importance is claimed and with what accuracy the existence of the secondary stresses has been proved. Assuming for a moment, that all the assertions and calculations of the examiners are correct, namely, that under certain conditions, happening perhaps once in 18 years, there are bending strains in the cables amounting to 30,600 lbs. per sq. in., which added to the direct tension in the wires produce an aggregate stress of 71,000 lbs. per sq. in., what will be the consequence to the safety of the structure? The examiners knew, or should have known, from different printed records that the minimum strength of the wire in the cables of the Brooklyn Bridge is supposed to be 172,000 lbs. per sq. in., with a minimum elastic limit of 66%, or 113,500 lbs. per sq. in.; hence, that a stress of 71,000 lbs. is considerably less than two-thirds of the elastic limit. The actual average strength of the wires, according to test records in my possession, is 177,000 lbs. per sq. in., and the actual elastic limit, or yield point, is 87.3%, according to tests recently made by the well-known test expert, Mr. Gus. C. Henning, with wire from the Brooklyn Bridge cables, which has been in my possession since 1878. The true elastic limit of the wire, therefore, is about 153,000 lbs. per sq. in., and a stress of 71,000 lbs. is less than one-half of the elastic limit.

In one part of the report the examiners explain the term "elastic limit" of steel or iron to be that load or force which, though much smaller than the true ultimate strength of the iron, will break the latter if the force be applied millions of times. For this reason, it is prudent for engineers not to allow the admissible stresses of structural iron to exceed two-thirds of its elastic limit. With this explanation, anybody can reason out, that, as far as the secondary stresses are concerned, the life of the Brooklyn Bridge might last a million of years, or, practically an infinite length of time. I do not wish that any engineer shall look at this statement as a joke, or as an exaggeration. It is not meant to be so. I make it in full seriousness, as a plain scientific inference from the figures and explanations given by the examiners themselves. Under these circumstances I cannot understand how the examiners could reasonably lay so much importance on harmless secondary stresses, and how they could justly make them the cause for predicting the destruction of the bridge and for urging "immediate repairs and alterations" to avoid an imaginary danger!

As far as the public and the safety of the bridge is concerned, the subject of secondary stresses might, herewith, be dismissed; but engineers may probably wish to know whether these stresses really exist, and if they do exist, how large they are.

When the examiners positively declare that "these stresses (meaning 71,000 lbs. per sq. in.) are 18% in excess of what is a permissible working stress;" that "the condition, therefore, is a very serious one," and, that "the most urgent improvement is that of cables in the center of main span," it is no wonder that the report created a general uneasiness and fear among the public which even spread beyond the Atlantic. One should suppose that nobody would make such serious charges against the safety of a structure and the ability of its builders without the best of reasons, based on exact facts and figures; but the examiners did not hesitate to base their charges on the following unstable grounds: "The computations on which our conclusions are based are necessarily incomplete and inexact;" "the stress in the cables is much above the safe limit, but with its exact amount unknown;" "the nature of the design makes it impossible to calculate the secondary stresses in the cables with the desirable degree of accuracy;" "the combination of circumstances causing the maximum secondary stresses is very improbable!"

The self-confessed inaccuracies, as quoted above, are sufficient to stamp all the conclusions of the examiners as unreliable, but I will add two more reasons with which I intend to prove that those conclusions are not only unreliable, but positively wrong, namely:

(1) The conditions assumed by the examiners as the base for their theory is entirely at variance with the reality; and,

(2) The theory itself is uncertain and doubtful, and in one respect decidedly wrong.

To understand the arguments for proving the last two assertions, I will briefly repeat the examiners' theory as described in the report:

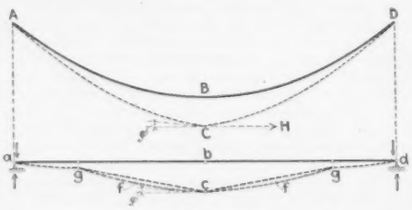



Fig. 2.

Referring to Fig. 2, let A B D be the position of the cable supposed to be a true parabola, at an average temperature of about 55° F., and loaded with the dead weight only. The line a b d represents the stiffening truss, supposed to be held at a and d, to have a hinge in the center at b and another hinge, or point of contrary flexure at g. If the cable be loaded with the maximum load at the time of the highest temperature, it will assume the position A C D, where A C and B D are parts of two different parabolas. The truss will assume the position a—g—c—g—d, supposing that it does not change its original shape of a straight line and that it merely turns around the hinges g and c. The total drop, b c, of the truss is equal to the drop, B C, of the cable, which is supposed to be 2.75 ft. The point g was chosen arbitrarily and placed near the end of the stay system, about 370 ft. from c. The drop of point, g, was assumed to be 0.75 ft., hence the vertical distance between g and c, designated $\delta$, is equal to 2 ft. When the cable elongates and deflects, each point of it between A and C sinks relatively a little more than each point of the straight truss g—c. The result of this circumstance is that the cable is relieved of part of its load which must be carried by the truss, changing the shape of the latter from the straight line g—c to the curved line g—f—c. The elastic flexure of the truss is supposed to be the same as if it were loosely supported at the ends g and c, and loaded with a uniform, but unknown, load p per lineal foot, which would cause a reaction at g and c, equal to $p \frac{l}{2}$, if l be the length of the line g—c. A tangent drawn to the curve g—f—c at point c forms with the horizontal line an angle $\varphi$, which is determined by the usual formula for the elastic line and expressed by the equation:

$$\tan \varphi = \frac{\delta}{l} - \frac{p\, l^3}{24\, E\, I}. \qquad (1)$$

where I is the moment of inertia of the truss and E the modulus of elasticity of steel. For determining p another equation is necessary which is found by simply assuming that the angle of a tangent to the cable at point C is equal to $\varphi$ and that the vertical component of the total tension in the cable at point C is equal to the reaction $p \frac{l}{2}$.

This vertical component is expressed by: H tan $\varphi$; if H be the horizontal tension in the cable; hence we have

$$H \tan \varphi = p \frac{l}{2}, \qquad (2)$$

and

$$\tan \varphi = \frac{p\, l}{2\, H}, \qquad (2')$$

This expression for tan $\varphi$ substituted in the first equation, we have:

$$\frac{p\, l}{2\, H} = \frac{\delta}{l} - \frac{p\, l^3}{24\, E\, I}, \qquad (3)$$

from which $p \frac{l}{2}$, or the end reaction of the trusses in the center of the span, was found to be 35.34 tons, and tan $\varphi$ = 0.00272. The angle $\varphi$ indicates the degree of curvature of the cable at point C, or the arc produced by the bending of the cable; hence if r be half the diameter of the cable, the elongation of the lower wires, and contraction of the upper wires, caused by the bending, is expressed by: r tan $\varphi$.

By other, more or less arbitrary and uncertain, assumptions, in regard to the tightness of the wrapping and factor of friction, the examiners find that the elongation r tan $\varphi$ must take place in 28½ ins., and, therefore, that it produces a fiber stress of 28,600 lbs. per sq. in. The bending of the individual wire to the same curvature produces a local bending stress of 2,000 lbs. per sq. in. making a total of 30,600 lbs. per sq. in., to be added to the direct tensile stress of the cable. Considering that the latter is only 32,400 lbs. for the whole dead weight of the bridge, the suspicion suggests itself that something must be wrong in this calculation, when the mere summer temperature combined with the existence of a hinged stiffening truss should have the same severe effect upon the cables as the addition of about 6,400 tons, uniformly distributed over the middle span of the bridge!

## ERRORS IN EXAMINERS' ASSUMPTIONS AND THEORIES.

This suspicion is well founded, considering that the whole calculation is based on a series of arbitrarily chosen assumptions either differing from the reality or being uncertain in themselves.

1. The examiners consider the slip joints in the trusses to act like hinges, which is correct for very small deflections, but decidedly incorrect for deflections as large as occur in the Brooklyn Bridge.

Theoretically, a hinge is perfect for transmitting all shear forces, but unable to resist even the smallest bending moment, while a slip joint can transmit only vertical shear forces, but is able to resist, though not perfectly, large bending moments.

Practically, a hinge, when the trusses deflect, causes a circular motion of the ends of the truss chords, at the top approaching to and at the bottom retreating from each other; while a slip joint causes a nearly horizontal motion of the truss chords, and forces them to remain parallel with each other.

This practical difference between hinge and slip joint is very important, because a hinge forms a sharp corner in the deflected truss, and is the supposed cause of secondary stresses in the cables; while a slip joint prevents the trusses from forming a corner and acts nearly the same as if the trusses were continuous. Necessarily there must be a little play between the tongues and the sleeves of the slip joint, and it is justifiable to consider a slip joint like a hinge to the extent of that play, but not beyond it.

The slip joints of the Brooklyn Bridge were constructed with a machine fit, which means that the play does not exceed $^1/_{16}$-in. The length of the tongue is 70 ins., hence the tangent of the possible angle formed by the hinge-quality of the slip joint is 0.0625 ÷ 70 = 0.00089. An increase of this angle can only take place by bending the tongues of the slip joints. This bending could be calculated exactly if we knew the forces acting on the plates. Not knowing these forces, I will, for the sake of argument, assume a weight of 36 tons as found by the examiners to be concentrated on the suspenders next to the slip joint in consequence of the deflection of the trusses. There are 36 slip-joint plates, hence the bending force on one plate would be about 2,000 lbs. To calculate the amount of flexure, the plate must be considered as a beam 86 ins. long, fastened at one end and loosely supported at the other, supporting a weight of 2,000 lbs. at a distance of 70 ins. from the loose end. The maximum deflection of such a beam with a rectangular section of 7¾ x 1⅝ ins., is, according to well-known formulas, 0.00228-in. at a point 49 ins. distant from the loose end; hence, the tangent of the angle is

$$\frac{2 \times 0.00228}{49} = 0.000094.$$

This added to the angle found for the play in the sleeves of the slip joint, make the tangent of the total angle,

0.00089 + 0.000094 = 0.000984-in.

or 0.000492-in. for the angle formed by the trusses with a horizontal line.

Under the examiners' assumptions the tangent of the same angle was calculated to be 0.0027, or over five times larger. The result of this investigation is that the

proper consideration of the slip joints, as compared with a hinge, would reduce the secondary stresses from 30,000 lbs. to about 6,000 lbs. per sq. in.

2. The examiners assume the point of contraflexure in the trusses at a distance of 370 ft. from the center of the span for the reason that this point coincides about with the end of the stay system. While the stays have certainly an influence upon the exact location of the point of contraflexure, there is no particular reason for assuming that it must be at the end of the stay system. On the contrary, there are two good reasons for supposing that the point of contraflexure is considerably nearer to the tower. First, because there is a slip joint in the top chord of the trusses, 565 ft. from the center of the span; and, second, because the outer group of stays between 379 ft. and 519 ft. from the center of the bridge, were adjusted with very little tension, supporting only about one-tenth of the dead weight of the superstructure, and, therefore, offering but little resistance to the deflection of the trusses. For these reasons it is probable that the point of contraflexure of the trusses is nearer to 470 ft. than 370 ft. from the center of the span. The examiners calculate that a difference of 70 ft. in the location of the point of contraflexure makes a difference of 7 tons in the value of $\frac{p\,l}{2}$; hence, if this point moves 100 ft. towards the tower the value of $p\,\frac{l}{2}$ would be about 26 tons in place of 36 tons, and the secondary stresses would be reduced in the same proportion; that is, in place of 6,000 lbs. they would be about 4,300 lbs.

3. The examiners assume that the unknown load p, of which the cable is relieved, and which is carried by the stiffening truss, is uniform per lineal foot, and that it produces an end reaction: $\frac{p\,l}{2}$. If the truss were supported only at the ends, this would be true, but it is highly improbable, if the truss, as in reality, is supported on 50 points, and each support is elastic. If the drop of the cable and that of the truss were the same at each point, there would be no load, p; no reaction, $\frac{p\,l}{2}$; and no secondary stress. It is only the difference between the drops of cable and truss which relieves the cable of a certain weight and causes an individual elastic deflection in the truss. This difference is largest about midway between the tower and the center of the span, and diminishes in both directions, becoming zero at the tower and at the center. It is, therefore, probable that the load p is not uniform, but is a function of the difference in the drop of cable and truss; and it is very improbable that all the loads, p, should accumulate and be transferred to the end suspender without leaving any part of the load on the intermediate supports. In fact, I consider it impossible that one suspender next to the slip joint should carry 9 tons more than the adjoining suspender which is only 7½ ft. distant from the former. The stiffness of the truss alone is sufficient to make such a difference impossible, unless it be created artificially, in which case several of the suspenders adjoining the end suspender would be relieved of nearly all of their regular load. I believe that most engineers will agree with me that the examiners' assumption of the end reaction being $\frac{p\,l}{2}$ (which means no reaction at all on any of the intermediate supports) is entirely hypothetical; hence, all results based on this assumption are unreliable. If the value of $\frac{p\,l}{2}$ is smaller than found by the examiners, the secondary stresses must be reduced still more.

4. The solution of the problem, according to the examiners' theory, is based on the assumption that the angle formed by the two half trusses at the hinge is equal to the angle formed by the two half parabolas of the cable at the lowest point of the bend.

If the truss be eminently rigid and the cable absolutely flexible, admitting a true corner at its vertex, I believe that this assumption is approximately correct, but it becomes decidedly wrong as soon as the cable ceases to bend in a sharp corner. Now, everybody knows that a sharp corner is impossible; not even a quarter-inch plate can be folded double without showing a curvature with a tolerably large radius. In other words, the tangent to the vertex of the deflected cable is always horizontal, and, therefore, is *not* equal to the angle $\varphi$ formed by the two half trusses. Of course, somewhere, at a certain distance from the vertex, a tangent to the cable will form the angle $\varphi$, but this distance is unknown and depends on the diameter and stiffness of the cable. The examiners admit that the cable does not form a sharp corner at its vertex, but they fail to take this circumstance into consideration in determining the angle $\varphi$, as shown by equations (1) and (2), quoted above. It needs no proof to assert that the curvature of the cable depends on its diameter and its inherent stiffness, but according to the examiners' equations the angle $\varphi$ would remain the same no matter whether the diameter of the cable be 1 ft. or 4 ft.; whether it consists of loose wires or of tightly wrapped wires. The theory, therefore, is in this respect incomplete, and the results obtained are doubtful.

The diameter of the cable is considered by the examiners only for determining the amount of elongation or contraction, due to the curvature of the cable as expressed by the angle $\varphi$; and the stiffness of the cable is considered only for determining the length of wire in which the elongation must take place. These proceedings, when applied to cables of different diameters lead to somewhat absurd results.

Let in Fig. 3 A O B represent the section of the Brooklyn Bridge cable. Assuming, for the moment, the calculations of the examiners to be correct, then the uniform primary stress in the cable is 40,400 lbs. per sq. in.; the secondary stress at A is + 28,600 lbs.; at B it is — 28,600 lbs.; and the fiber stress in the individual wire is 2,000 lbs. Now, I propose to cut one inch from the cable section at top and bottom. This will make no change in the formulas of the calculation, but it will reduce the area of the cable from 145 sq. ins., as assumed by the examiners, to 137.08 sq. ins., increasing proportionately the primary stress from 40,400 to 42,730 lbs. On the other hand, the secondary stress will be reduced at D and E in the proportion of 6.75 : 7.75, and assume the value 24,910 lbs., in place of 28,600 lbs.; and the stress in the individual wire will be 1,740 lbs. in place of 2,000 lbs. We have, therefore, the following comparison:

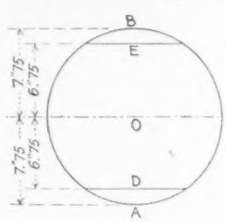


Fig. 3.

| | Examiners' figures; actual cable section. | Experimental figures; reduced cable section. |
|---|---|---|
| Primary stress, per sq. in..... | 40,400 lbs. | 42,730 lbs. |
| Secondary stress, per sq. in... | 28,600 " | 24,910 " |
| In individual wire, sq. in.. | 2,000 " | 1,740 " |
| Total ..... | 71,000 lbs. | 69,380 lbs. |

In other words, by making the cable weaker we make it stronger!

It would be difficult to find anybody to believe this result, but such is the fact, if the examiners' theory be correct, and if the fiber stress in the extreme wires of the cable be a true criterion for the safety of the cables, as the examiners claimed it to be when they declared the Brooklyn Bridge to be in imminent danger!

5. To find an equation for determining the reaction $\frac{p\,l}{2}$ it was necessary to equate it with the vertical component of the tensions of the cable at a point where the tangent to the cable forms the angle $\varphi$ with the horizon. The correctness of this assumption is very doubtful, and I believe it is wrong. I have explained that the load p has nothing to do with the absolute loads on the cable and merely depends on the difference between the deflections of cable and truss, therefore it is principally a function of the moment of inertia of the truss. On the other hand, the horizontal tension H in the cable is a direct function of all the loads on the same. For instance, if the dead weight of the bridge were doubled, it would increase the tension H, but it would not make a particle of difference in the forms of the cables or trusses when they drop 2¾ ft. from their original position. Hence, all the formulas of the examiners' theory would be precisely the same as quoted above, only the absolute value of H would be different and therewith tan $\varphi$ and the secondary stresses would also be different. As the latter are caused *only* by *bending* the cable, and not by loading it, the formula, which gives different values for the *same* bend under different loads, must necessarily be wrong.

6. The final result for the secondary stress was obtained by the assumption of a certain friction between the wires of the cable. This is such an uncertain factor, known by nobody, that no reliance can be placed on any figures. By assuming a certain tension in the wrapping wire the examiners found the friction to be 1,333 lbs. per sq. in. per lin. in. If the examiners had assumed twice or only half that amount, nobody could contradict it with better reason than to doubt the given figure. However, the probability is that the given figure is too large, because experience has proved that the wrapping wire at the bottom of the cable under each cable band sounds hollow to the tap with a hammer.

In the preceding six paragraphs I have shown that the calculation of the secondary stresses in the cables, as given by the examiners, is based on conditions different from the reality; on a wrong theory, and on an unknown factor of friction; hence, it contains so many elements of inaccuracy that the result cannot be considered much better than a guess, which will dwindle down to insignificant proportions when some of the inaccuracies are corrected.

THE CORRECT ASSUMPTIONS AND CALCULATIONS.

This discourse would be incomplete if, after having pointed out the errors in the examiners' views and calculations, I should not show a more exact method for determining the effects of moving loads and temperature upon the form of the cable and the stresses it is subjected to. To carry out the calculations with full exactness is very laborious, hence I shall only indicate the method and be satisfied with approximate results in exemplifying the method.

First, it is necessary to calculate the precise form of the cable curve at medium temperature, and under a distribution of dead load which corresponds with the reality. Next, the precise form of the cable must be calculated for the maximum temperature and with the live load added. The difference in deflection between the two curves must be noted at a number of points, and must be compared with the deflections of the floor curve from the higher to its lower position. The two differences will not agree, which indicates that the cable has been relieved of a certain weight at some points which is carried by the stiffening truss and partially transmitted again to the cable at other points. With a new assumption of load distribution the cable curve must now be calculated again, also the elastic deflection of the truss under the loads of which the cable was relieved. If the distance between these curves is equal to the correct suspender lengths at each point the assumption of load distribution was correct; if not, another assumption must be made and the calculation be repeated until the differences in cable deflection and floor deflection are the same. The results obtained in this way are as exact as it is possible to calculate anything where the moments of inertia and the modulus of elasticity play a part.

The following is a reproduction of the original conditions and calculations on which the form and the stresses of the cables of the Brooklyn Bridge were determined:

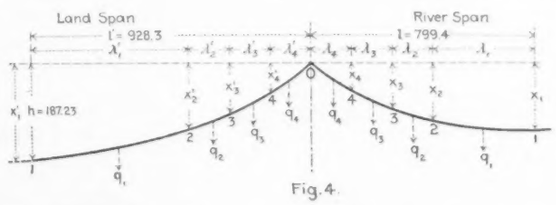


Fig. 4.

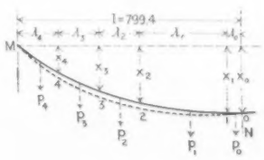


Fig. 5.

Fig. 4 represents one land span and half the river span of one cable, subjected per lineal horizontal foot, to the dead loads $q_1$ $q_2$, etc., which are supposed to be uniform over the lengths $\lambda_1$ $\lambda_2$, etc.

The difference in the values of $q_1$ $q_2$, etc., is due to the effect of the stays which were calculated to sustain most of the dead weight near the towers, and very little of it at their outward extent.

The exact theory for determining the form of the cable and the stresses under the conditions as illustrated in Fig. 4, is fully described in my treatise on cable making published by Van Nostrand in 1877. I will, therefore, merely indicate the method of procedure for this calculation. Each part of the cable curve, being subjected to a uniform load over its horizontal projection, is necessarily an arc of a parabola which has its imaginary vertex at a certain unknown distance from the point of intersection O and at a certain unknown height below the same.

Referring first to the river spans and calling the horizontal distances of the different vertices from point O, A, B, C, the vertical distances a, b, c, and the ordinates of the points where two parabolas join: $x_4$, $x_3$, $x_2$ and $x_1$ we have, therewith, ten unknown quantities to determine, for which we need ten equations. These ten equations are found by expressing in formulas the following conditions:

| | Equations. |
|---|---|
| 1. That each parabola goes through the respective junction point = ..... | 3 |
| 2. That the tangent to each parabola at the junction point is the same..... | 3 |
| 3. That the horizontal tension is the same in each parabola ..... | 3 |
| 4. Equation expressing the length of the curve.. | 1 |
| Total ..... | 10 |

The second condition presupposes that the curve is continuous without abrupt changes in curvature, which, in reality, is the nature of a wrapped-wire cable. The correctness of this original theory and of the results obtained by the calculations has, recently, been fully corroborated by the present engineers of the bridge. They plotted an exact survey of the cable in conjunction with the curve as determined by calculation, and both lines were found to be precisely parallel to each other.

For the river span we assumed the following numerical values:

| | | | |
|---|---|---|---|
| $\lambda_4$ = 140 ft. | $q_4$ = 771 lbs. | (for one cable). | |
| $\lambda_3$ = 140 " | $q_3$ = 1,491 " | " " " | |
| $\lambda_2$ = 140 " | $q_2$ = 1,816 " | " " " | |
| $\lambda_1$ = 379.4 " | $q_1$ = 2,051 " | " " " | |

l = 799.4 ft.

If the length, S, of half the cable be known the ordinate $x_4$ can be calculated, or $x_1$ might be assumed, and the length calculated.

We assumed S = 812.665, and found from the solution of the ten equations:

| | |
|---|---|
| A = 1,749.7655 ft.; | a = 246.4585 ft. |
| B − $\lambda_4$ = 832.4141 " ; | b = 145.7275 " |
| C − $\lambda_4$ − $\lambda_3$ = 568.4964 " ; | c = 132.3705 " |

$x_4$ = 37.8011 ft.
$x_3$ = 71.0931 " Horizontal tension, H = 4,788,900 lbs.
$x_2$ = 97.5577 " Maximum tension, T = 4,975,400 "
$x_1$ = 128.3818 " Tan $\alpha$ at point O = 15° 44'.

The exact useful area of one cable is 147.6 sq. ins.; hence, stress in center of span: $\frac{4,788,900}{147.6}$ = 32,400 lbs. per sq. in.; stress at tower: 33,700 lbs. per sq. in. The land span was calculated to balance the horizontal tension of the river span, and we assumed:

$\lambda'_4$ = 140 ft.
$\lambda'_3$ = 140 "
$\lambda'_2$ = 140 "
$\lambda'_1$ = 508.3 "

l' = 928.3 ft.

h = 187.23 ft.; H = 4,788,900 lbs.

The results from the solution of the equations are:

| | |
|---|---|
| $x'_4$ = 47.2413 ft. | Length of curve, S' = 952,117 ft. |
| $x'_3$ = 89.8568 " | Maximum tension, T = 5,210,000 lbs. |
| $x'_2$ = 125.8219 " | Hence, |
| $x'_1$ = 187.3900 " | max. stress = 35,300 lbs. per sq.in. |

The above figures, calculated at the time the bridge was designed, are the original results of which the correctness was strikingly corroborated by the recent survey of the cables.

The live load, on one cable, as found by the examiners, is 625 lbs. per lin. ft. If the stays carry the same proportion of live load as they do dead weight, the load on the cable will be, referring to Fig. 4:

| | |
|---|---|
| $q_1$ + 625 lbs. = | 2,051 + 625 lbs. |
| $q_2$ + 553 " = | 1,816 + 553 " |
| $q_3$ + 454 " = | 1,491 + 454 " |
| $q_4$ + 235 " = | 771 + 235 " |

Let us consider now the main span (see Fig. 5), subjected to dead and live load, under the following conditions:

(1) An increase of temperature of 50° over mean temperature;

(2) An aggregate of 36,000 lbs. removed from the load over the length $\lambda_1$ and one-half of it added to the load over $\lambda_0$, and the other half added to the load over $\lambda_2$.

(3) A deduction of 50% of the live load over the length $\lambda_2$ and one of 40% over $\lambda_3$, on account of the greater sustaining power of the stays at a high temperature;

(4) A drop of 2.7 ft. of the cable in center of span from its position at mean temperature.

These conditions correspond substantially with those assumed in the calculations of the examiners, except the distribution of the loads which is here according to the reality, while the examiners assumed all loads to be uniform throughout.

The second condition, a concentration of 18 tons distributed over only 6 ft. in the center of the span, was chosen not because it might possibly occur, but merely for comparison with the examiners' results, which were based on analogous conditions.

We have, therefore, referring to Fig. 5, the following values:

| | Pounds. | Feet. |
|---|---|---|
| $p_0$ = 2,051 + 625 + $\frac{18,000}{3}$ | = 8,676. | $\lambda_0$ = 3 |
| $p_1$ = 2,051 + 625 − $\frac{36,000}{376}$ | = 2,580. | $\lambda_1$ = 376.4 |
| $p_2$ = 1,816 + 553 − $\frac{553 \times 50}{100}$ + $\frac{18,000}{140}$ | = 2,342. | $\lambda_2$ = 140 |
| $p_3$ = 1,491 + 454 − $\frac{454 \times 40}{100}$ | = 1,764. | $\lambda_3$ = 140 |
| $p_4$ = 771 + 235 | = 1,006. | $\lambda_4$ = 140 |

$x_0$ = 128.3818 + 2.7 = 131.0818 ft.

The method of calculation is the same as shown above, only in this case there are 13 unknown quantities, and 13 equations to be solved. The results are (see Fig. 5):

| | | |
|---|---|---|
| $x_4$ = 36.9633. | A = 1,625.8127 | |
| $x_3$ = 71.1718. | B − $\lambda_4$ = 891.1451 | a = 224.2830. |
| $x_2$ = 98.5924. | C − $\lambda_4$ − $\lambda_3$ = 565.7643 | b = 155.1185. |
| $x_1$ = 131.07521404. | D − $\lambda_4$ − $\lambda_3$ − $\lambda_2$ = 386.4884 | c = 134.4006. |
| $x_0$ = 131.0818. | S = (length of curve) 812.9981. | d = 131.0974. |

H = 5,928,000 lbs. = 40,100 lbs. per sq. in.
$T_{max}$ = 6,149,000 " = 41,600 " " "

Comparing these figures with those found for the dead weight at mean temperature we find that the difference in the length, S, is 812.9981 − 812.665 = 0.3331. The elongation of the cable by a rise of 50° of temperature is 0.3248 ft., and 0.22 ft. through the increase of tension; hence, the new length should be: 812.665 + 0.3248 + 0.22 = 813.2098, or 0.211 ft. longer than the value found by calculation. This indicates that the curve as represented by the above given figures is not correct, and that it should occupy a position about as shown in a dotted line. Such a curve can only be obtained by changing the distribution of loads; diminishing $p_0$ and $p_1$, and increasing $p_2$, $p_3$ and $p_4$. If we compare the values of $x_4$, we find that point 4 in the second case is 0.9 ft. above point 4 of the first case. This indicates that the concentration of a heavy load in the center of the span has the tendency to raise the cable near the towers. Considering that the suspenders at and near point 4 are attached to a strong cantilever of about 120 ft. length, it will require a great force to bend this truss $^9/_{10}$ of a foot upward. In other words, the resistance offered to the cable prevents it from rising and adds an artificial load to $p_4$ and $p_3$ which will lower the cable, as shown in the dotted line, to correspond with the correct length of 813.2098 ft. It is evident that the bending stress of the cable at N is smaller for the dotted line than for the full line. I shall, therefore, not correct the calculation, but will determine the bending stress for the full line, which will give an exaggerated and for my argument more unfavorable result, than the correct line would have given.

To find the bending stress it is necessary to determine the radius of curvature in the center of the span for the high and low position of the cable. In the first case we have a parabola of the equation: $\lambda_1^2 = 2\,p\,(x_1 - x_2)$ (p being the parameter of the curve)

$$2\,p = \frac{\lambda_1^2}{x_2 - x_1}.$$

To find the ordinate f at a distance of 3 ft. from the vertex, we have

$$3^2 = \frac{\lambda_1^2}{x_2 - x_1}\,f;$$

$$f = \frac{3^2 \times 30.8241}{379.4^2} = 0.0019272.$$

The radius of a circle (see Fig. 6) drawn through this ordinate f and the vertex is expressed by the formula:

$$r = \frac{m^2 + f^2}{2\,f} = \frac{3^2 + 0.0019272^2}{0.0038544}$$

= 2,334.90473 ft.

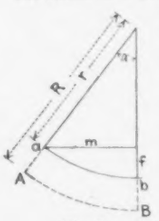


Fig. 6.

$$\tan \alpha = \frac{m}{r - f};$$

$\alpha$ = 0° 4' 25.05" = 265.05";

length of arc

$$a - b = \frac{2\,r\,\pi \times 265.05}{360 \times 60 \times 60}$$

= 3.00052827 ft.

If $\overline{a\,b}$ represents the center or neutral line, and $\overline{A\,B}$ the length of the outer fiber of the cable, we have for the latter:

$$\overline{A\,B} = \frac{2\,R\,\pi \times 265.05}{360 \times 60 \times 60} = 3.00135755,$$

when R = r + half diameter of cable = r + 0.6458333 = 2,335.640563. Difference in length between $\overline{A\,B}$ and $\overline{a\,b}$ = 0.00082928 ft. In the second case, referring to Fig. 5, we found f' = $x_1 - x_0$ = 0.00658596, and, in the same manner as shown above we find:

r' = 683.275;
R' = 683.92083333;
$\alpha'$ = 0° 15' 5.64";
$\overline{a'\,b'}$ = 3.00053488;
$\overline{A'\,B'}$ = 3.00286754;
$\overline{A'\,B'} - \overline{a'\,b'}$ = 0.00233266.

Therefore, the actual elongation of the outer wire, from its original length, will be:

0.00233266 − 0.00082928 = 0.00150338 ft.

The tension necessary for producing this elongation is:

$$T = \frac{0.00150338 \times 28,000,000}{3.00135755} = 14,020 \text{ lbs per sq. in.}$$

Calculating in the same way, the bending stress of an individual wire of 0.177-in. diameter which, from an original circle of 2,335 ft. radius, is bent around a circle of 683 ft. radius, we find T = 212 lbs.; hence, the total bending stress will be 14,232 lbs., in place of 30,000 lbs. as found by the examiners under the same conditions. If the bending stress be reduced in accordance with the correct curvature of the cable, it is safe to say that the true bending stress will be about one-half to one-third of the stress as found, under the same conditions, by the examiners with the help of a defective and uncertain theory.

Considering, furthermore, as explained previously, that the conditions assumed are contrary to the true and more favorable facts, we can finally say that, similar to Mr. Wasell's alarming primary stresses in the cables of the Niagara bridge, the alarming secondary stresses of the examiners dwindle down to insignificant proportions.

## PRESSURES ON TOWER MASONRY.

The examiners have calculated that the direct pressure on the tower masonry combined with the bending stress, under the assumption that the saddles are immovable and that the middle span be loaded with the maximum load while the land spans are free from moving load, will amount to 39 tons per sq. ft., at the base of the center shaft. This calculation is probably correct, but there is nothing alarming in it. When the bridge was designed the same calculation was made and the maximum pressure on the masonry was found to be 30 tons per sq. ft. under the supposition that the saddles were movable. This pressure was considered perfectly safe for the high-class masonry of which the Brooklyn Bridge towers are constructed.

The assumption of fixed saddles was based by the examiners on the fact that, "under hasty observations," they did not notice any motion in the saddles. This is very natural and would be the case if the saddles were in the best of condition, because the moving loads on the bridge are generally balanced in the different spans. If the middle span alone were loaded with the maximum load, a proposition which in itself is very improbable, I am confident that the saddles *will* move. Evidence of this assertion was given three years ago when a bottom chord buckled, which threw suddenly a large load, previously carried by the stays, on the cables and caused the south saddles to move 3 ins. towards the river. Afterwards the saddles moved partially back, showing that the saddles *are* movable when there is force enough to overcome the resistance. Another instance was furnished by the Cincinnati bridge, where the saddles had not moved for many years and where, during the reconstruction of the bridge, sand and dust was taken out of the saddles by the shovelful and where direct evidence showed that all the rollers were not parallel with each other. In spite of this great resistance the saddles moved readily when a considerable preponderance of weight on one side or the other furnished sufficient power to overcome all resistance.

We can, therefore, justly assert that, even under a very unnatural and improbable supposition, the greatest pressure on the masonry will never exceed 35 tons, and, as a rule, will be considerably below 30 tons.

Of buildings in which the pressure is near or above 30 tones per sq. ft. I mention:

| | |
|---|---|
| Hotêl des Invalides, Paris | 38 tons. |
| St. Peter's, Rome | 25 " |
| Church of St. Méry | 41½ " |
| Pantheon, Paris | 41½ " |
| Column of All Saints at Angères | 55 " |

It is doubtful whether the masonry of these buildings, which have stood a century or more, is as good as that of the Brooklyn Bridge. In any case it cannot be better because the masonry of the bridge towers is the strongest and most durable that possibly can be made.

## ALTERATIONS SUGGESTED BY EXAMINERS.

In order to reduce the imaginary excessive tension in the cables, and the pressure on the tower masonry, about which the examiners are unnecessarily alarmed, the latter propose the following alteration in the superstructure of the bridge:

1. To increase and strengthen the stay system and to remove all suspenders within the stay system.

2. To anchor the land cables at a point near the end of the stay system, to a pier or to the ground.

To carry out these two alterations would, in my opinion, be the most fatal error that could be made. Not only would it not reduce the maximum stresses, but it would increase them, and the supposed imperfections in the present construction would be aggravated.

The examiners have offered no calculations to show in figures the advantages of the proposed alterations, claiming that the necessary calculations cannot be made without the knowledge of the length of the land cables. I happen to know this length, and I have made the calculations with the above stated result.

In order not to lengthen this discourse too much I will not repeat the computations, which are lengthy and complicated, in detail, but merely will indicate the method of calculation and give the final results.

Relieving the cables of all weight within the stay system would cause the vertex of the cable to drop several feet, which is not admissible, indicating that the length of the cable in the main span must be shortened. To find the correct length, I refer again to Fig. 4, and calculate the form of the cables under the condition that $\lambda_4$ = 420; $\lambda_3$ = 0; $\lambda_2$ = 0; $\lambda_1$ = 379.4; $q_4$ = 500; $q_3$ = 0; $q_2$ = 0; $q_1$ = 2,051 lbs.; $x_1$ = 128.3818 ft. The solution of the equations with these data gives the following results:

Horizontal tension, H = 4,039,000 lbs. = 27,300 lbs. per sq. in.;

Length of half the curve: S = 811.8343.

The original length was found to be 812.665 ft., hence, the difference between the two lengths, or 0.8307 ft., must be taken from the main span cable and added to each land cable. On account of the decrease in tension both cables will contract, 0.1538 ft. in the main span, and 0.1803 ft. in the land span, making a total contraction of 0.3341 ft. Therefore, the length to be added to each land cable is only 0.8307 − 0.3341 = 0.4966 ft.; the original length of the land cable was 952.117 ft.; hence, the new length of the land cable is 952.6136 ft. It can easily be seen that under the dead load alone, distributed in the same way as in the main span, the tension in the land cable is much smaller than the tension in the main span; hence, in order to balance the latter, a new force must be introduced. This is furnished by the holding down anchors, which I

assumed to be at a distance of 420 ft. from the center of the tower. In the problem to be solved we know the length of the curve; the horizontal tension, and the loads, while the holding down force P, and the deflection d of the cable at the point where the force P is applied, are unknown quantities. For determining the unknown force P and unknown deflection d, we have two equations—one expressing the length of the curve, and the other the horizontal tension. The solution of the two equations is not difficult but quite complicated, hence I give merely the results as found, namely: P = 62.6 tons; d = 127.5 ft. The component of P, perpendicular to the cable is 56.8 tons. This is for mean temperature.

Let us consider, now, a fall of 50° in temperature. The original length of the cable in the main span will decrease 0.325 ft. and the horizontal tension will increase to 4,093,000 lbs, or 27,700 lbs. per sq. in. (as compared with 27,300 lbs. at mean temperature). The land cable, when anchored down, cannot contract without producing an immense tension; hence, the cable will elongate under the increased tension to the same or nearly the same amount as the fall of temperature would contract the same, which requires a tension of 11,600 lbs. per sq. in. The horizontal component of this tension is 11,000 lbs., which, added to the former 27,300 lbs., makes a total stress of 38,300 lbs. per sq. in.

In our original calculation we found the horizontal stress under the dead load to be 32,400 lbs. per sq. in., which will increase to 33,000 lbs. for a fall of 50° F. We see, therefore, that the proposad alteration, while diminishing slightly the stresses in the main span, cause an increase of 14% (5,300 lbs. per sq. in.) in the stresses of the land cables every time the thermometer registers 5° above 0; and, of course, more yet, if the thermometer sinks lower.

The direct horizontal tension at 5° F. will be 4,039,000 + 1,623,600 = 5,662,600 lbs., which compared with the main span tension of 4,093,000 lbs., leaves a preponderant pull of 1,569,600 lbs. per cable towards the land, or an aggregate of 3,136 tons on the whole tower. This is for dead load only, and under the examiners' assumption that the saddles are fixed.

Under the supposition that the land spans are loaded with the maximum live load, while the main span is free from live load, the horizontal tension in the land cables will become about 400 tons greater than if subjected to dead load only; hence, there will be an additional pull on the tower of about 1,600 tons, which, together with 3,136 tons as given above, will raise the bending stresses in the towers to alarming proportions. The holding down force, which for mean temperature was 62.6 tons, will at 5° F., become about 86 tons, and the component, perpendicular to the cable, about 78 tons. When a concentrated load of 18 tons in the center of the main span causes secondary stresses, though not as alarming as the examiners found them, but still of considerable magnitude, we can imagine what these secondary stresses will come to, if the concentrated load is more than quadrupled.

I think I have shown that the alterations proposed by the examiners will not only be no "improvement," but a decided detriment to the safety of the bridge. Anchoring the land cables down will: (1) Increase the direct or primary tension of the cables; (2) increase the bending stresses of the towers; (3) increase the secondary stresses in the cables. All of these results were known to the engineers of the Brooklyn Bridge at the time when a similar plan of holding the land cables down was in contemplation as a feature of the design, and was abandoned when an investigation made all the disadvantages manifest.

Still, the examiners boldly assert that the designer of the Brooklyn Bridge knew nothing about temperature stresses or secondary stresses.

It is true he knew nothing of stresses which did not exist, nor was he acquainted with the fancy modern nomenclature, but he knew that "temperature strains" and "bending strains" should be avoided as much as possible. All the Roebling bridges are manifestly designed in all details with the object of avoiding concentrated loads and consequent bending strains, and with means for distributing the loads uniformly in order to reduce the direct tension.

## STRENGTH OF THE FLOOR SYSTEM.

A part of the examiners' report is devoted to the floor system with which they find considerable fault, claiming that, contrary to good bridge practice, excessive and dangerous stresses exist in different members of the floor. In the criticism of the floor system and the constant comparison with railroad bridges and with what is considered "standard," the examiners show more plainly than in anything else that their experience has been solely with truss and cantilever bridges. The builders of truss bridges may have found by experience that it is safer and more economical to make certain details stronger than what is theoretically necessary, and to use extra precautions in certain constructions and in inspections to prevent accidents, which has established the "standard" referred to by the examiners; but it does not follow that the same standard must be applied to structures with different conditions.

I claim that suspension bridges have a right to their own standards, which has also been established by the experience of men, who for half a century have made a specialty of the study and of the construction of such bridges. Of course, any part that is absolutely unsafe or weak must be condemned in any structure, but where it is a matter of possible contingencies, a suspension bridge is justified in taking greater risks than a truss bridge. While a mishap on a suspension bridge has either no consequence at all, or is confined to one spot and to few persons, a similar mishap on a truss bridge might be the cause of a wholesale calamity, together with the destruction of the entire bridge. I have spoken already of this peculiarity of suspension bridges in the paragraph about wind stresses, and have explained that the lateral stiffness of a truss bridge is an absolute necessity for its existence, while in a suspension bridge it is merely a convenience and comfort. The same rule holds good in the construction of other parts. Take, for instance, the spacing of ties and the application of guard rails. There is no more danger of a rail breaking or of a locomotive becoming derailed on a bridge than on any other part of a railroad. Still, it is customary on railroad bridges to lay the ties very close and to provide the track with guard rails. It is easy to see why this is done. While an accident to a train when happening on the open road may be calamitous for a number of people, it means, when happening on a bridge that it may also be fatal to the bridge itself. There is certainly no objection to using the same precautions on suspension bridges, but the necessity of doing so is less urgent, because no local accident endangers the general safety of a suspension bridge. For this reason I believe that the examiners' general criticism on the floor system is unjustified, because they compare everything with the standard of railway truss bridges for which the conditions are entirely different from those on the Brooklyn Bridge. Since the stringers, ties and rails have carried the loads safely for 18 years, and since no derailment has ever happened, there is certainly no cause for alarm when the construction differs more or less from that customary in truss bridges.

The calculation of the stresses in the different steel beams of the floor system agrees in some instances exactly with the calculations of the builders of the bridge, and in other instances they differ so widely that there either must be a misunderstanding of the construction, or that the "usual way of calculating" disregards the more intricate theories which must be adapted to the existing facts.

The examiners find the stresses in the main floor beams to be 15,500 lbs. per sq. in., which is correct; but by no means too much, as the examiners claim. It is, on the contrary, a very safe stress and, for the superior class of material used in the Brooklyn Bridge, it is proportionately, smaller than what commonly is allowed for ordinary bridge steel in railway bridges. According to actual tests, made with full sized specimens, the steel has an elastic limit of 50,000 to 55,000 lbs. per sq. in., and an ultimate strength of 75,000 to 80,000 lbs. per sq. in. Engineers will admit that, under these circumstances, a unit stress of 15,000 or 15,500 lbs. per sq. in. is very conservative, and perfectly safe.

The same unit stress is supposed to exist in other parts of the floor construction, but the examiners found a stress of 23,000 lbs. in the intermediate floor beams, and one of 34,000 lbs. per sq. in. in the intermediate chord channel of the outer high truss on which the intermediate floor beam is supported.

I have taken occasion to recalculate the stresses in the supporting channel under an assumption of concentrated loads as given by the examiners for the heaviest cars, and my results show that the examiners are again mistaken in their conclusions.

To prove this assertion, I shall give, in the following, a detailed description of the method of calculation:

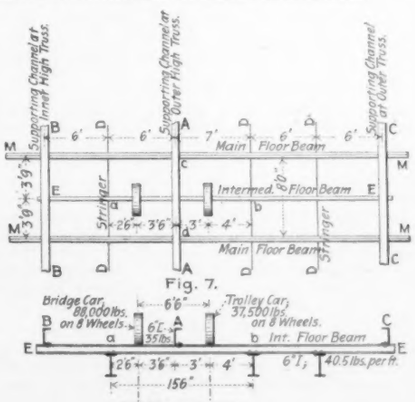



Fig. 7.

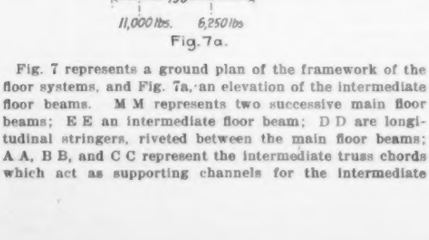



Fig. 7a.

Fig. 7 represents a ground plan of the framework of the floor systems, and Fig. 7a, an elevation of the intermediate floor beams. M M represents two successive main floor beams; E E an intermediate floor beam; D D are longitudinal stringers, riveted between the main floor beams; A A, B B, and C C represent the intermediate truss chords which act as supporting channels for the intermediate floor beams. The latter are continuous girders over five openings, while the supporting channels are continuous for the whole length of the bridge, being riveted to the truss posts at the intersection of every main floor beam. The intermediate floor beams, as well as the supporting channel, can, therefore, be considered as girders fastened at the ends, the former at a, b and A; the latter at c and d. Both girders must act in unison and the strength of one cannot be calculated without considering the strength of the other.

The greatest live load to which either intermediate floor beam or supporting channel can be subjected is shown in Fig. 7, by one wheel of a bridge car representing 11,000 lbs., and one wheel of a trolley car, representing 6,250 lbs. Any adjoining wheel of either car comes beyond the main floor beams or stringers, and has no effect upon the girders in question.

Consider, first, the intermediate floor beam to be disconnected from the channel and to have a clear span of 156 ins. between points a and b. If subjected to a load of 1,284 lbs. under the bridge car wheel and to one of 716 lbs. under the trolley car wheel, the beam will deflect 0.0481-in. at or near the point A. If the two loads are concentrated in the center of gravity which is 58 ins. from point a, and 98 ins. from point b, the maximum deflection is expressed by the formula:

$$\frac{2{,}000}{E\,I} \quad \frac{2 \times 58^3 \times 98^3}{3\,(58 + 3 \times 98)^4} = 0.0481\text{-in.}$$

when E = 28,000,000, and I = moment of inertia of 6-in. I-beam = 23.705.

Now, if it can be shown that the channel A A deflects, under its maximum load, the same amount as the I-beam does, the two girders work in unison. The load of 2,000 lbs. is absorbed in bending the intermediate floor beam, and, therefore, can be subtracted from the original wheel loads, which will be 11,000 − 1,284 = 9,716 lbs; and 6,250 − 716 = 5,534 lbs., respectively. These loads are transferred, proportionate to the distances, on the supporting channel at a point midway between d and c and on the adjoining stringers at points a and b, as follows.

| | Lbs. |
|---|---|
| Proportion of bridge car on supporting channel | 4,048 |
| Trolley car on supporting channel | 3,162 |
| Dead weight of floor for width of 3½ ft. and a length of 40 ins. | 540 |
| Total | 7,750 |

The supporting channel weighs 35 lbs. per ft. and has a moment of inertia of 14.84, hence the deflection at the middle point is expressed by the formula:

$$\delta' = \frac{7{,}750 \times 80^3}{192 \times 28{,}000{,}000 \times 14.84} = 0.0497\text{-in.}$$

This is a little more than we found for the deflection of the intermediate floor beam, which indicates that the true load, supported by the channel, is less than 7,750 lbs. The fiber stress in the supporting channel for a load of 7,750 lbs., concentrated in the center of its span, is expressed by

$$S = \frac{7{,}750 \times 80}{8 \times W} = 15{,}800 \text{ lbs.,}$$

when W is the moment of resistance $= \frac{14.84}{3}$.

The exact stress is probably not over 15,500 lbs. and this result shows that the examiners abnormally exaggerate when they claim a stress of 34,000 lbs.; and it is also a test of how much reliability can be placed on the examiners' figures wherever they find "excessive" stresses! It is not necessary to give more examples.

## THE SAFETY OF THE BRIDGE.

*The Brooklyn Bridge is safe!* But, in view of the fact that on two occasions certain details of the structure gave way, it is demonstrated that all parts of the bridge, which the examiners had no time to investigate, should be thoroughly inspected, and that provisions should be made to avoid occurrences similar to those of last summer and three summers ago. In my letter to the Engineering News of Aug. 8, 1901, I pointed out that these accidents are not significant of a general weakness, but are merely local disturbances which need not cause any alarm. In the old Monongahela bridge at Pittsburg, suspender rods broke daily without ever causing an accident, and in the Niagara bridge, after the old wooden truss had been replaced by a new steel stiffening truss, it happened that the top chord not only buckled, but completely tore asunder, which merely frightened the engineer of the passing train, but otherwise had no appreciable influence on the stiffness of the bridge. This proves, what I have repeatedly asserted—that stiffening trusses are rather luxuries than necessities, and that all labor was wasted which the examiners expended in measuring and calculating the stiffening trusses of the Brooklyn Bridge with a view of determining whether the bridge be safe or not. In this connection I beg to say that some points of the stiffening trusses which should have been well inspected were neglected by the examiners. I mean the slip joints, about which the examiners merely report that "they work well," without metnioning that they are badly worn, and, in place of fitting snugly, have a play of 5/16 to 9/16 of an inch. There is no doubt that, in this worn condition, the junctions of the trusses in the center of the span act more like hinges than slip joints, and the examiners should

have recommended an immediate repair, in order to avoid or reduce the dreaded secondary stresses in the cables.

This discourse is not intended to discuss different suggestions or to offer plans for improvements which would prevent the accidents as mentioned above. I refer to my article in the Engineering News of Aug. 8, where some suggestions for the alteration of some details have been outlined. Outside of these, unless it be the intention to increase the general capacity of the bridge, no extensive alterations are necessary, and to "let well enough alone," will be the best plan!

CONCLUSIONS.

Summing up the results of the foregoing comments on the examiners' report in connection with my explanations and calculations, we find:

(1) That the Brooklyn Bridge is well preserved and not overloaded, and, therefore, safe, according to the examiners' own statements.

(2) That the Brooklyn Bridge was always safe; contrary to the examiners' statements; because there are no serious defects in the design; there are no serious secondary stresses in the cables, and there are no excessive stresses in any other part of the bridge.

(3) That the alterations, proposed by the examiners, for increasing the safety of the Brooklyn Bridge, are not only unnecessary, but also technically objectionable, inasmuch as they would diminish the safety of the cables in place of increasing the same.

In conclusion, I beg to repeat that the principal object of this discourse is to restore the proper public confidence in the safety of the Brooklyn Bridge. In doing so I was frequently obliged to criticise the work and the report of the examiners, which was done without personal feelings and without doubting the professional ability of the examiners, but, in defense of the cause I represent, and in refutation of the rebukes to the engineers of the Brooklyn Bridge, which the examiners felt called upon to add to their condemnation of the structure itself.

For a number of years it has been the habit of some engineers to criticise the Brooklyn Bridge, and similar suspension bridges, sometimes justly, but more often through a misconception of the construction; but never before has it happened that two engineers have attacked a great structure and its builders with equal severity and with less justice than it was done by the examiners in the case of the Brooklyn Bridge. According to their report there is not a single good feature in that structure; everything is bad; no credit is given to the originator of the design and to the builders, for what they have accomplished; but, on the contrary, they are reprimanded and sharply censured. With a few strokes of the pen the examiners have created almost a panic among the traveling public; they have undermined the general confidence in the safety of a structure of which this community used to be proud; they have spread distrust and doubts to other communities possessing similar structures, and therewith they might have injured the reputation of the engineers of the bridge, if the latter had not established, by numerous other works, their standing among the profession.

On what grounds are all these attacks and assertions based? They are not based on precedents, because all Roebling bridges have done service for from 40 to 45 years, which is longer than any other modern bridge can boast of, and, what is important to capitalists, all Roebling bridges were financially successful, proving that they were built safely and economically; they cannot be based on the past experience of the examiners, because that is principally with other types of bridges; they are not based on observations because the examiners found the bridge well preserved; the attacks are not justly based on calculations because the latter are confessedly incomplete and inexact, and, as I have shown, they are results of uncertain and partially incorrect theories. In other words, the attacks are based on misunderstanding or on imagination, as there is no tenable reason for any part of the examiners' report which attempts to prove the unsafety of the bridge.

For these reasons I have tried to be explicit in my principal arguments against the examiners' assertions, and I offer the necessary minuteness in some details as apology for the length of this discourse.

The matter is serious and important enough to deserve study and a thorough examination. I submit, herewith, this reply to the examiners' report to the profession at large for judgment, whether attacks on a structure, not sustained by unmistakable proofs, will, in future, be approved of as a fair criticism, or whether such proceedings will be condemned as unjust and unscientific.

Wilhelm Hildenbrand.

New York, Jan. 6, 1902.

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NEW YORK STATE HIGHWAY IMPROVEMENT is reported upon by State Engineer and Surveyor Bond for the past year. During the four years 1898-1901 the sum of $670,000 has been appropriated for good roads. The aggregate result obtained is represented by 25 improved highways built in 12 counties, with a total length of 60 miles; and 32 roads, aggregating 110 miles in length, in 16 counties, now under contract and the necessary money appropriated. Petitions have been received from 40 counties for 298 roads, aggregating 1,282 miles in length, and surveys have been made for over 624 miles of these roads in 32 counties. In addition to the roads completed or under contract, surveys and plans and estimates have been made and accepted by boards of supervisors for 87 roads, aggregating 244 miles in length, and the counties' one-half of the estimated cost, or $916,051, has been appropriated by the supervisors in 21 counties. Surveys for roads aggregating 314 miles have been made and will be submitted to boards of supervisors at their coming sessions.

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EXPERIMENTS WITH WIRELESS TELEPHONY are now being carried on by a number of inventors. A recent press dispatch reports a demonstration of wireless telephony given by a French engineer named Ducretet on Jan. 7. The transmiting and receiving instruments were about 100 ft. apart, and it is said that there was no difficulty in conversing even in low tones by means of the apparatus.

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AMERICAN METHODS AND ENGLISH TRADE unions lately came into conflict in Manchester, England. The British Westinghouse Electric & Manufacturing Co. is building extensive works at that place, and James Stewart & Co., of St. Louis, Mo., are the contractors for erection. The building and equipment are to cost about $7,000,000, and the English builders estimated that the erection of the buildings would require five years' time. They have been built in less than a year by the American firm named. British workmen were used; but instead of laying 450 bricks in a nine-hour day—the trade union figure—each man laid an average of 1,800 bricks per day, with 2,500 on very plain work. This result was brought about by the persistent effort of Mr. G. C. Stewart, the manager of the American contractors. By a system of rewards to foremen, the elimination of poor workmen, daily reports of progress from each foreman, etc., he quickly worked the men up to the averages quoted. The delegates of the trade unions threatened trouble, but as Mr. Stewart paid more than union rates, and plainly told the delegates that he would either run the job or employ non-union men, his methods prevailed, with the result noted.

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THE JESSOP STEEL CO., of Washington, Pa., has been lately incorporated, to further the plan of Wm. Jessop & Sons, the long-established English manufacturers of crucible steel, to carry on the manufacture in this country. The initial plant will include a large crucible melting shop, a rolling mill plant and a power-house and sheet steel will first be made, using largely Swedish iron ore. These works were designed by James J. Mahon, of Chicago, who is the engineer in charge of construction and equipment. Contracts for all except the machine tools, cranes, etc., have been placed. The directors of the company include Wm. Jessop, President of Wm. Jessop & Sons, of Sheffield, England; Herbert Hughes, Secretary of the Sheffield Chamber of Commerce, and director in Wm. Jessop & Sons; W. F. Wagner, General Manager of the New York Offices of the British Company; E. L. Hand, of Philadelphia; C. N. Brady, of Washington, Pa., and Sydney J. Robinson, Managing Director of Wm. Jessop & Sons.

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A CIVIL SERVICE EXAMINATION for subinspector of buildings in the Bureau of Yards and Docks at a salary of $4 per day will be held at the navy yard, League Island, Pa., on Jan. 21, 1902.

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THE BIG BEAVER MEADOW-QUAKAKE TUNNEL is practically completed; the headings from the two ends having met on Jan. 6. This tunnel is nearly 1½ miles long, and will drain the coal mines of Coxe Bros. & Co., in the Beaver Meadow basin. It was commenced about two years ago.

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FOR A SUCCESSFUL AIRSHIP, the Louisiana Purchase Exposition management will offer a prize of not less than $100,000, says a St. Louis item. A tournament of airships is proposed, and full details of conditions are to be published later.

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THE GODAVARI RIVER BRIDGE, on the East Coast Railway in India, has been recently completed. This bridge has 50 spans of 150 ft. each, and is 9,096 ft. long over all. The piers were sunk by the pneumatic process and by ordinary well-sinking methods. The engineer is Mr. F. T. G. Walton, M. Inst. C. E.

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THE CARNEGIE INSTITUTION AT WASHINGTON is to have the following purposes—according to an official announcement lately made by Mr. Chas. D. Walcott, Secretary of the corporators of the institution: (1) To increase the efficiency of universities, etc., in the country by utilizing and adding to their existing facilities and by aiding in experimental work. (2) To discover the exceptional man in any department of study, and to aid him financially in pushing his especial work. (3) To promote original research. (4) To increase the facilities for higher education. (5) To enable students who locate at Washington to avail themselves of the advantages of the collections, etc., there gathered together in the several government departments. (6) To insure the prompt publication and distribution of the results of scientific investigation. To this end Mr. Carnegie proposed to place $10,000,000 in 5% bonds with the following board of trustees: Ex-Officio, the President of the United States; the President of the U. S. Senate, the Speaker of the House of Representatives; the Secretary of the Smithsonian Institution; the President of the National Academy of Sciences; Grover Cleveland, New Jersey; John S. Billings, New York; William N. Frew, Pennsylvania; Lyman J. Gage, Illinois; Daniel C. Gilman, Maryland; John Hay, District of Columbia; Abram S. Hewitt, New Jersey; Henry L. Higginson, Massachusetts; Henry Hitchcock, Missouri; Charles L. Hutchinson, Illinois; William Lindsay, Kentucky; Seth Low, New York; Wayne MacVeagh, Pennsylvania; D. O. Mills, California; S. Weir Mitchell, Pennsylvania; W. W. Morrow, California; Elihu Root, New York; John C. Spooner, Wisconsin; Andrew D. White, New York; Edward D. White, Louisiana; Charles D. Walcott, District of Columbia; Carroll D. Wright, District of Columbia.

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PHILIPPINE RAILWAY DEVELOPMENT, commenced under government aid, is strongly urged in the annual report of the Philippine Commission. In the island of Luzon about 1,000 miles of line would meet all reasonable demands for years to come, and this would cost about $35,000,000; and in Mindanao, now practically unexplored, 500 miles would probably meet immediate demands. The Commission suggests that the government of the Philippines be permitted to grant special charters for railways and to guarantee interest on the investment; but not to grant public lands. The Commission believes that a low rate of guaranteed interest would invite competition among bidders, secure low rates for construction and traffic, and an absolutely certain return on the investment in the near future. The objection is the burdening of the island revenue with a fixed charge, of say $1,650,000, or 3% on $55,000,000. But it believes that many of the lines would pay from the start and that all of them would ultimately be self-supporting.

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THE COLLEGE OF FORESTRY, connected with Cornell University, is defended by Director B. E. Fernow in answer to misstatements made recently in the New York "Times." Instead of "denuding" 500 acres of Adirondack land, it has "cut over" this area; and it planted 255 acres out of 300 acres requiring replanting, though some of the "cut over" territory can not be planted the same year. The purpose of the forester is so to arrange this whole 30,000 acres that it can be harvested continuously; growing a young crop and cutting out old trees every year. The reproduction of wood crops, and earning a revenue therefrom is the purpose for which the college was organized. While the hunter's interests are of secondary consideration in the very small part of the Adirondacks effected, Mr. Fernow claims that the change from open old timber to a young plantation is rather an advantage to the breeding of deer.

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A BRITISH COAL COMMISSION has been appointed to inquire into the coal resources of Great Britain. The similar commission of 1865-71 reported that the supply of coal to a depth of 4,000 ft. was 90,236,000,000 tons. Since 1871 and up to 1900, inclusive, 5,025,000,000 tons of coal have been mined, an output far in excess of the rate of exhaustion assumed by the royal commission. The British experts now say that if this rate of doubling the output in 30 years is kept up the end of the 20th century will see the exhaustion of England's coal fields.

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UNDERGROUND PUBLIC WATER CLOSETS in Union Square, San Francisco, are proposed. The Merchants' Association of that city has secured plans for the structure from Mr. Edw. R. Swain, a local architect. Mr. L. M. King, Superintendent of the association, has made an illustrated report (see Merchants' Association Review, November, 1901) on the subject. The estimated cost of a double station, with six closets on the women's side, six closets and ten urinals on the men's side, and a wash basin on each side, is $5,793, or $6,845, according to the interior finish. The report and plans have been approved by the directors of the association. The city park commissioners favor the scheme, provided the necessary money can be secured.

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THE APPROXIMATE COST OF RAILWAY TRACK, per mile, in the four years 1898-1901, is set forth in a detailed tabular form by Mr. Francis How, certified public accountant, of 6 Wall St., New York. This table shows that the aggregate cost of construction is now less than in 1899; or, taking a 90-lb. rail as an example, the total cost per mile for each of the four years named, is $6,799.48, $10,469.00, $8,394.26 and $8,736.30 respectively. For a 70-lb. rail, these costs are $5,545.36, $8,566.20, $6,795.84 and $7,071.50 respectively. The table specifies the quantities required of rails, spikes, angle plates, nuts, etc., ties, track-laying and ballast; and it gives these items for 56, 60, 65, 70, 80 and 90-lb. rails. This table will be furnished on application to Mr. How.